

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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CORROSION OF CONCRETE

By JOHN R. BAYLIS,* Assoc. M. Am. Soc. C. E.

To Be Presented May 5, 1926

SYNOPSIS

Portland cement does not form compounds insoluble in water corrosive to calcium carbonate. The solubility of calcium carbonate depends on the alkalinity, hydrogen-ion concentration, and other salts in the water.

Some of the generally assumed compounds of calcium and silica may not exist in cement. It is possible that the excess of calcium in concrete over that theoretically required to combine with certain proportions of silica is attributable to surface adsorption or to a solid solution. Finely ground hydrated Portland cement gives off calcium hydroxide approximately to the point of calcium hydroxide saturation. The saturation equilibrium decreases by the addition of successive quantities of water, forming a curve that does not indicate definite high calcium compounds of silica.

When mixed with water the finer particles of cement tend to coagulate around larger particles of cement or aggregate, producing a structural formation somewhat similar to a meshwork of minute fibers filling the space between the larger particles. The hydration of cement probably follows the general law of the chemical changes of solids, namely, that a solid does not change in chemical composition without first going into the soluble state. In the case of Portland cement the solution and reprecipitation apparently takes place without any material change in the shape or size of the individual cement particles, except that the particles actually unite in the newly formed solid where they touch each other.

Porosity is a very important factor in determining the life of concrete exposed to water or to the weather. It largely governs the rate of diffusion of soluble compounds from the interior to the surface. Water molecules apparently are oriented when they are against any solid surface. This is the orderly arrangement of the molecules in such a manner that they do not move freely with the molecules somewhat distant from the surface. Such a phenomenon in engineering is called surface friction. When two solids are submerged with part of their surfaces close together the oriented layers of water may join, and there is little movement of the water molecules between the surfaces. If all surfaces are so close together that the oriented water layers unite through-

NOTE.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in August, 1926, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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out, there is little communication of the water in the interior with the outside surface. This is the ideal condition for resisting corrosion.

A method of measuring the voids in concrete is given in this paper. It is suggested that the No. 8 sieve be the dividing line between fine and coarse aggregate.

Changes taking place in concrete exposed to water or to the weather are usually the liberation of calcium hydroxide, and its combination with carbonic acid to form calcium carbonate. If the water is corrosive to calcium carbonate it will be dissolved gradually, but the rate at which it goes into solution is slow. The gelatinous compounds of alumina and silica remaining after the calcium has been dissolved greatly aid in decreasing the solution rate as the diffusion through such compounds is slow. Freezing or considerable surface friction hastens the rate of corrosion, largely by removing these gelatinous compounds.

Moisture evaporating from a concrete surface tends to concentrate destructive compounds, if present, at various points. The surface of most of the concrete exposed to water corrosive to calcium carbonate should be water-proofed.

Structures valued at many millions of dollars are now showing deterioration where they are exposed to water and freezing weather. In many instances the deterioration has progressed to a point where it is jeopardizing the structures, and millions of dollars will have to be spent on repairs and replacements within the next decade to keep them in serviceable condition. This should make the problem of concrete corrosion of great interest. Engineers have been too ready to place all the blame for disintegration on poor construction, and have not fully realized that concrete has its limitations. It is the purpose of this paper to point out a few causes for disintegration when good materials have been used. In some instances the causes are of such a complex nature that it will require years of patient research to fully explain them, but in most instances there is a similarity suggesting a common cause. It is hoped that these few fundamental facts that have been established will be a start in the solution of this important problem.

It seems essential to use one chemical term that may not be understood by engineers. The symbol, pH , is used to represent the concentration of dissociated hydrogen-ions in the water. Pure water dissociates slightly into positively charged hydrogen-ions (H^+) and negatively charged hydroxyl-ions (OH^-), which bear a certain ratio to each other. The addition of acid or alkaline compounds changes this ratio. Pure water at a pH of 7 has approximately 0.0001 parts per million of dissociated hydrogen-ions. At a pH of 8 the hydrogen-ions are only one-tenth the amount at a pH of 7, and at a pH of 9 only one one-hundredth the amount. A pH of 6 has ten times as many dissociated hydrogen-ions as a pH of 7. The pH has a very important bearing on the saturation equilibrium of certain compounds.

Corrosion as used in the paper is distinguished from disintegration in that it is a weakening of the concrete by the dissolving of certain compounds from

the cement. Disintegration of weak concrete may be caused by freezing when there may have been no dissolving or changing of the cement compounds.

SOLUBILITY OF MOST OF THE COMPOUNDS IN CEMENT

Heretofore engineers have assumed that Portland cement formed compounds insoluble in water. It will be shown that this is not the case. The fact that there is usually an intermediate change to calcium carbonate makes the problem, in effect, one of the solubility of calcium carbonate plus the protective action of certain gelatinous compounds.

As shown by the equilibrium curves in Fig. 1 the solubility of calcium carbonate depends on the alkalinity and hydrogen-ion concentration of the surrounding solution. Certain salts in solution effect this equilibrium as shown by the two upper curves, for 10% solutions of sodium chloride and sodium sulfate. For conditions represented by points above the curves, calcium carbonate will be precipitated; for those below, it will be dissolved. These equilibriums were established at room temperature, and the hydrogen-ion concentration (pH), determined colorimetrically.

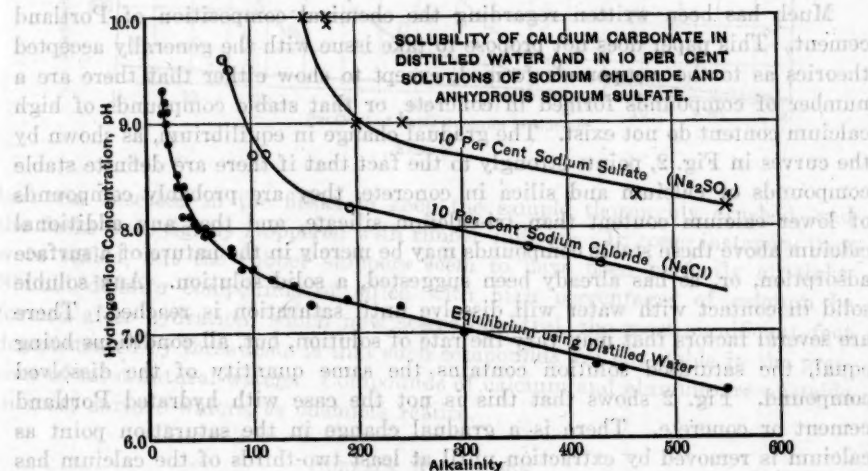


Fig. 1.

The procedure in obtaining these curves was as follows: For the determination of the curve without neutral salts about 50 grammes each of calcium carbonate from three different sources was pulverized in a laboratory mortar and put into 300-cu. cm. glass-stoppered bottles. Chemically pure calcium carbonate, pulverized limestone, and the surface crystals from a saturated solution of lime water were the three substances. Both distilled water and water from the city supply of Baltimore, Md., was used; it was changed several times before the first results were recorded. High alkalinity concentrations were obtained by passing carbon dioxide through the solution. The bottles containing the calcium carbonate were filled with water, the glass

stoppers inserted, and then the specimens were allowed to stand in the laboratory for one week with an occasional shaking. Later experiments have been run in which pyrex flasks were used and more than a month allowed for solubility equilibrium to become established. The same general procedure was followed for establishing the curves where neutral salts were present.

The rate at which reactions (such as the dissolving or precipitating of calcium carbonate) take place near the curve may be extremely slow, but as the distance increases the rate of solution or precipitation increases rapidly. Calcium carbonate solubility equilibrium might be called one of the fundamental forces of Nature, and it is the real key to concrete corrosion under many conditions of exposure. From the fact that rain water has practically no alkalinity and a pH usually less than 6.0, it is readily shown by Fig. 1 how corrosive to concrete it may be. Nearly all surface waters will dissolve calcium carbonate, but the rate is quite variable. More than 90% of the total calcium in concrete will be dissolved by water slightly corrosive to calcium carbonate. The compounds remaining are usually soft and of a gelatinous nature.

CHEMICAL COMPOUNDS AS GENERALLY ASSUMED MAY NOT EXIST

Much has been written regarding the chemical composition of Portland cement. This paper does not propose to take issue with the generally accepted theories as to the compounds formed, except to show either that there are a number of compounds formed in concrete, or that stable compounds of high calcium content do not exist. The gradual change in equilibrium, as shown by the curves in Fig. 2, points strongly to the fact that if there are definite stable compounds of calcium and silica in concrete, they are probably compounds of lower calcium content than tri-calcium silicate, and that any additional calcium above these stable compounds may be merely in the nature of a surface adsorption, or, as has already been suggested, a solid solution. Any soluble solid in contact with water will dissolve until saturation is reached. There are several factors that may vary the rate of solution, but, all conditions being equal, the saturated solution contains the same quantity of the dissolved compound. Fig. 2 shows that this is not the case with hydrated Portland cement or concrete. There is a gradual change in the saturation point as calcium is removed by extraction until at least two-thirds of the calcium has been extracted.

While the amount of alkali in solution, as shown by the curves of Fig. 2 and by Tables 1, 2, 3, and 4, is expressed in parts per million of calcium carbonate as determined by the ordinary alkalinity test, most of it is in the form of calcium hydroxide with probably just a little sodium and potassium hydroxide or carbonate present for the first few extraction tests. To obtain the parts per million of calcium oxide, the alkalinities should be multiplied by 0.56. The saturation point of calcium hydroxide at a temperature of 60° Fahr. is approximately 1 340 parts per million of calcium oxide (CaO), or, expressed in terms of alkalinity as used in the diagrams and tables, about 2 400. Any material showing an alkalinity of more than 2 400 probably contains a more soluble alkali such as sodium or potassium, or contains a super-

saturation of calcium hydroxide. Most of the tests in Fig. 2 were made after the specimen had stood from 10 days to 2 weeks with frequent agitation. Solubility equilibrium is not quite reached in this time; a few check tests were made in which from 2 to 4 months were allowed. The longer periods of standing do not materially alter the shape of the curves. All material was thoroughly pulverized to the point where it could pass a 100-mesh sieve. A steel

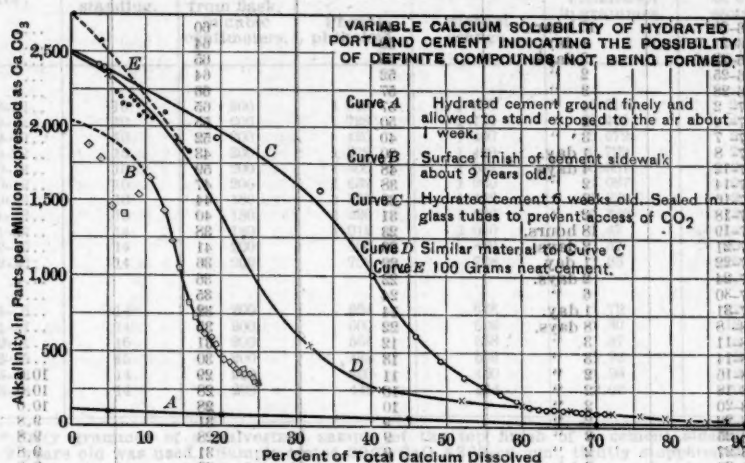


FIG. 2.

ball was inserted in the flasks to keep the sample thoroughly crushed, and the flasks were tightly stoppered with rubber stoppers. In some instances they were sealed, but the rubber stoppers seem to have been perfectly air-tight. Should definite compounds of silica with high percentages of calcium be formed after hydration, which now seems doubtful, the most significant fact demonstrated by these tests is that such compounds are not stable in the presence of most natural waters. Compounds of calcium and alumina are unstable in most surface waters, as chemists realize.

SURFACE ADSORPTION

Surface adsorption is a force almost unknown to engineers. Many, if not all, solids have the power of attracting certain compounds to their surface; that is, the surface molecules form some kind of a loose chemical combination with certain other compounds. The concentration of the attracted molecules may not be limited to the thickness of one molecule. The amount of a compound which will be adsorbed depends on the surface area, the attractive force of the solid, the temperature, the concentration of the compound being adsorbed in the surrounding phase (solution or gas), and probably on other forces. The presence of compounds in the solution which are not adsorbed frequently affects the amount and rate of adsorption. It is possible for certain greatly expanded gelatinous compounds to hold by surface adsorption large quantities of other compounds.

TABLE 1.—EXTRACTION OF THE ALKALIES IN HYDRATED PORTLAND CEMENT.*

Date:	Time standing.	ALKALINITY, IN PARTS PER MILLION.		pH.
		Phenolphthalein.	Methyl orange.	
1924:				
6-20	16 hours.	49	60
6-21	1 day.	50	64
6-23	2 days.	55	65
6-25	2 "	52	64
6-28	3 "	57	66
7- 2	4 "	57	65
7- 4	2 "	50	55
7- 7	3 "	40	52
7- 8	1 day.	30	43
7-12	4 days.	48	59
7-14	2 "	38	47
7-16	2 "	33	44
7-18	2 "	31	40
7-19	18 hours.	23	38
7-21	2 days.	26	41
7-23	1 day.	22	36
7-24	2 days.	22	36
7-30	6 "	24	35
7-31	1 day.	14	32
8- 8	8 days.	22	35
8-11	3 "	12	31
8-14	3 "	13	30
8-16	2 "	11	29	10.0
8-18	2 "	10	28	10.0
8-20	2 "	10	28	10.0
8-22	2 "	9	31	9.8
8-25	3 "	9	28	9.8
8-27	2 "	9	31	9.8
8-29	2 "	9	28	9.8
9- 1	2 "	9	27	9.8
9- 5	4 "	9	26	9.8
9- 7	2 "	7	28	9.7
9- 9	2 "	6	26	9.6
9-11	2 "	6	25	9.6
9-13	2 "	5	27	9.6
9-21	6 "	7	29	9.7
9-23	2 "	5	24	9.6
9-25	2 "	5	24	9.6
9-27	2 "	5	24	9.6
9-29	2 "	4	23	9.4
10- 1	2 "	4	24	9.4
10- 3	2 "	4	24	9.3
10- 5	2 "	5	25	9.4
10- 7	2 "	5	25	9.3
10- 9	2 "	5	25	9.3
10-11	2 "	4	25	9.2
10-13	2 "	4	24	9.3
10-15	2 "	5	23	9.3
10-17	2 "	4	23	9.2
10-18	1 day.	3	23	8.9
10-19	1 "	3	23	8.9
10-20	1 "	3	21	8.9
10-21	1 "	2	21	8.9
10-22	1 "	1	19	8.5
10-23	1 "	1	19	8.5
10-24	1 "	1	20	8.4
10-25	1 "	1	18	8.5
10-27	2 days.	1	21	8.4
10-28	1 day.	1	19	8.5
10-29	1 "	1	19	8.8
10-30	1 "	0	16	8.0
10-31	1 "	0	13	6.9
11- 1	1 "	0	11	6.7

*A cement mortar, 1 month old, was pulverized very finely, and allowed to stand 1 week exposed to the air. A 2-gramme sample containing 1.19 grammes of CaO was used. The total CaO extracted, as calculated from the alkalinity tests, was 1.12 grammes. The residue at the end of the test contained 0.032 grammes of CaO.

TABLE 2.—EXTRACTION OF THE ALKALIES FROM OLD CONCRETE.*

Date:	Days standing.	Amount of solution withdrawn from flask, in cubic centimeters.	ALKALINITY, IN PARTS PER MILLION.		Total CaO withdrawn in grammes.	Percentage of total sample.
			Phenolphthalein.	Methyl orange.		
1924:						
8-30	10	200	1 840	1 578	0.210	2.10
9-9	10	200	1 728	1 770	0.408	4.08
9-19	10	200	1 420	1 460	0.572	5.72
9-29	10	200	1 372	1 400	0.729	7.29
10-9	10	200	1 500	1 532	0.901	9.01
10-19	10	200	1 632	1 660	1.067	10.67
10-29	10	180	1 396	1 432	1.230	12.30
11-8	10	180	1 220	1 236	1.355	13.55
11-22	14	180	1 012	1 040	1.47	14.7
12-6	14	200	800	820	1.57	15.7
12-20	14	200	708	728	1.65	16.5
1925:						
1-3	14	200	624	648	1.73	17.3
1-17	14	200	600	620	1.80	18.0
2-1	16	200	556	588	1.87	18.7
2-16	15	200	488	536	1.92	19.2
3-2	14	200	460	480	1.98	19.8
3-16	14	200	446	464	2.02	20.2

* Fifty grammes of a pulverized sample of the top finish of a cement sidewalk more than 9 years old was used. Sample tested 20% CaO A300-cu. cm., tightly stoppered flask was used. An iron ball was inserted in flask on October 9, 1924, in order to pulverize thoroughly the particles that had lumped together. Complete equilibrium probably was not reached in some of the tests before October 19.

TABLE 3.—EXTRACTION OF THE ALKALIES FROM PORTLAND CEMENT.*

Date:	Days standing.	Amount of solution withdrawn from flask, in cubic centimeters.	ALKALINITY, IN PARTS PER MILLION.		Total CaO withdrawn in grammes.	Percentage of total sample.
			Phenolphthalein.	Methyl orange.		
1924:						
10-26	13	900	1 982	1 944	0.98	16.3
11-6	11	800	1 544	1 576	1.69	31.6
11-24	18	800	952	988	2.13	39.9
12-11	17	885	584	612	2.43	45.7
1925:						
1-4	24	800	424	448	2.63	49.4
1-19	15	800	324	344	2.79	52.4
2-2	14	900	264	284	2.98	54.9
2-16	14	950	216	232	3.05	57.3
3-2	14	960	172	188	3.16	59.3
3-16	14	950	146	160	3.24	60.8

* Neat cement mortar sealed in glass tube for six weeks. Pulverized, and 10-gramme sample used. Cement mortar tested 53.32% CaO. One liter flask used.

TABLE 4.—EXTRACTION OF THE ALKALIES FROM PORTLAND CEMENT.*

Date.	Days standing.	Amount of solution withdrawn from flask, in cubic centimeters.	ALKALINITY, IN PARTS PER MILLION.		Total CaO withdrawn, in grammes.	Percentage of total sample.
			Phenolphthalein.	Methyl orange.		
1924:						
9-2	10	200	2 720	2 752	0.308	0.47
9-12	10	200	1 968	2 012	0.533	0.82
9-23	11	200	2 043	2 046	0.762	1.17
10-3	10	200	2 244	2 296	1.019	1.57
10-13	10	200	2 400	2 428	1.291	1.98
10-23	10	200	2 376	2 408	1.661	2.56
10-27	10	200	2 156	2 192	1.806	2.78
11-2	10	200	2 312	2 340	2.069	3.18
11-12	10	200	2 536	2 572	2.356	3.67
11-24	10	200	2 524	2 568	2.643	4.07
12-8	14	200	2 420	2 452	2.918	4.48
12-22	14	200	2 352	2 384	3.185	4.90
1925:						
1-5	14	200	2 300	2 324	3.445	5.30
1-19	14	200	2 264	2 288	3.701	5.19
2-2	14	200	2 228	2 260	3.954	6.08
2-16	14	200	2 192	2 220	4.203	6.48
3-2	14	200	2 168	2 192	4.448	6.83
3-16	14	200	2 128	2 156	4.689	7.22
3-31	15	200	2 124	2 148	4.936	7.58
4-14	14	200	2 168	2 208	5.182	7.98
5-3	19	200	2 160	2 188	5.428	8.36
5-17	14	200	2 124	2 156	5.670	8.73
5-31	14	300	2 086	2 064	6.016	9.26
6-23	28	300	2 072	2 104	6.350	9.81
7-12	14	300	2 040	2 074	6.728	10.35
7-26	14	300	2 040	2 074	7.071	10.87
8-9	14	350	2 056	2 092	8.184	12.54
8-23	14	900	1 876	1 896	9.139	14.04
9-6	14	900	1 804	1 840	10.066	15.48
10-18	42	900	1 428	1 456	10.800	16.62
12-20	68	900	1 204	1 228	11.419	17.55
1926:						
1-3	14	900	928	948	11.897	18.27

* 100 grammes of neat cement containing 65.0 grammes of CaO was used. Prior to November 12, 1924, the sample tended to form lumps. An iron ball was inserted in the flask and all lumps were crushed.

The usual conception of gelatinous compounds is that they are fairly soft, yet actually they may be composed of a network of minute chains of crystals, frequently sub-microscopic in size, which are as hard as the original compact solid. Surrounding these crystals, or any solid which is submerged, is a layer of water that has lost its power to flow. When the solid is exposed to the air there also appears to be a surface layer of air which does not flow. These are probably oriented layers, that is, there is an orderly arrangement of the molecules next to the surface.

The force with which adsorbed compounds (assuming that water or other liquids and gases are adsorbed) are held attracted to a surface is greatest next to the surface, gradually diminishing until a point is reached where the surface has no effect. Some gelatinous compounds are able to hold more than fifty times their weight of water, agar being such a compound. Water does not flow from the gel regardless of how long it may be left with unsupported sides; yet there is no impervious membrane holding the water back. It will evaporate if the moisture content of the air is below the saturation point,

but it will not flow away from the fibers of the gel. When pressure is applied it tends to overcome the attractive forces of the solid, so that materials impervious at a low pressure may not be so under high pressure. Clay may be nearly water-tight, yet have more than 50% of its volume in voids. Particles of the same shape as those composing the clay, but about 1 mm. in diameter, may arrange themselves so that the total percentage of voids is much less than in the clay, yet water will flow through them hundreds of times as fast as through the clay.

The fact that the water molecules may be adsorbed or oriented against a solid surface and thus prevented from flowing gives a good illustration of the power of solid surfaces. Orientation and adsorption are by no means confined to the water molecules, for the concentration on solid surfaces of molecules dissolved in water covers a wide range of compounds and conditions. When a solid has attraction for both acids and alkalis there is a certain hydrogen-ion concentration at which practically no acid ions are adsorbed, and another concentration at which practically no alkaline ions are adsorbed. Between these points both ions appear to be attracted to or near the solid surface in variable quantities, depending on the hydrogen-ion concentration and a number of other variables.

HYDRATION OF PORTLAND CEMENT

Chemical compounds probably do not change from one compound to another without first going into the liquid stage. Certainly this is the case with most changes, and future research will probably prove it to be true for practically all compounds. Most chemists now believe that there are exceptions to this rule, although every indication points to the fact that cement does follow the general rule. In the hydration of Portland cement the dissolving and reprecipitation as the cement compounds unite with the water takes place with little change in volume or form from that in which the particles of cement are left after they have been mixed with water. Where the particles touch, or are extremely close together, the dissolving and reprecipitation builds them up together. It is this that gives cement its binding power. The reaction progresses slowly, and possibly requires considerable time before it reaches the center of fairly large particles, if it ever does. Reground cement mortar several years old usually has a slight tendency to set when it has been mixed with water, indicating that the original reaction might not have been complete for all the particles.

A nearly saturated solution of calcium hydroxide tends to coagulate finely divided particles of many compounds, and when the particles of cement liberate caustic lime it, in turn, will partly coagulate the finer particles of cement around the larger particles, giving a structural formation somewhat as shown in Fig. 3. The full effect of this tendency is not realized in concrete when only sufficient water is added for proper handling, for the fine particles do not become greatly dispersed in the water. It is not possible to produce neat cement mortar with less than about 25% of voids when it is first mixed. The hydration does not reduce this percentage materially. It is only by the addition of some other compound, such as carbonic acid, which forms calcium

carbonate with the liberated calcium hydroxide, that the voids are reduced. The formation of calcium carbonate tends to build up another structural formation within the voids of the cement, greatly reducing them and usually adding strength to the concrete.

There is a point, however, long before all the voids are filled, where the stagnated layers of water or air close the pores against rapid communication with the surrounding water or air. Then, the concrete becomes water-tight or air-tight, yet it may have more than 20% of its mortar volume as voids. It is the formation of calcium carbonate within concrete that gives it the greatest protection from the weather, and this protection is largely the result of reducing the mass to the impervious or stagnated stage. The gelatinous aluminum hydroxide and silica offer the greatest protection to concrete submerged in water corrosive to calcium carbonate, but the intermediate change to calcium carbonate greatly aids in retarding the rate of corrosion for most natural waters. This applies to conditions in which there is nothing tending to remove the gelatinous compounds from the outer concrete surface.

STRUCTURAL FORMATION OF CEMENT MORTAR

The addition of water to cement, and the mixing, leaves it in a porous structural formation with a slight tendency to assume the characteristic mesh-work of fibers as shown in Fig. 3. This is from a very porous section of concrete, largely composed of the coagulated finer particles which have reprecipitated into solid fibers. The drawing is partly camera lucida. It is from a greatly expanded part of the mass, and was selected because it would be difficult to show clearly the more compact part. There are numerous large particles varying in size from those shown to others as large as the circle. A sand grain 0.5 mm. in diameter would be about twice the diameter of the circle.

A true picture of the structural formation of hydrated cement mortar would show a structure with fibers much shorter, thicker, more irregular in shape, and more compact than shown in Fig. 3, with some particles as large as the entire drawing.

Immediately after mixing with water every particle of cement touches or is extremely close to another particle, and usually to several others. A pile of crushed stone will probably give some idea of the arrangement of the cement grains; however, this is not a correct picture, for the finer particles of cement tend to coagulate around the larger particles of cement or aggregate somewhat as shown in Fig. 5. This applies to the finer particles of dirt or silt as well as to the cement. To obtain Fig. 5, a small particle of sand, rejected for use on the addition to the Montebello Filters in Baltimore, Md., was agitated in water and then allowed to stand about 30 min.; 1 cu. cm. of the dirty water was placed in a counting cell and a few particles of cement added. The water was slightly agitated for about 5 min., and the camera lucida drawing was made about 15 min. afterward. A gelatinous coating had formed around practically all the particles of cement, as shown in Fig. 5.

Using the pile of stone to complete the illustration of the action of the cement—the outside of the stones becomes plastic and adheres, and then solidifies again. The plastic part, representing the change from the oxides in cement

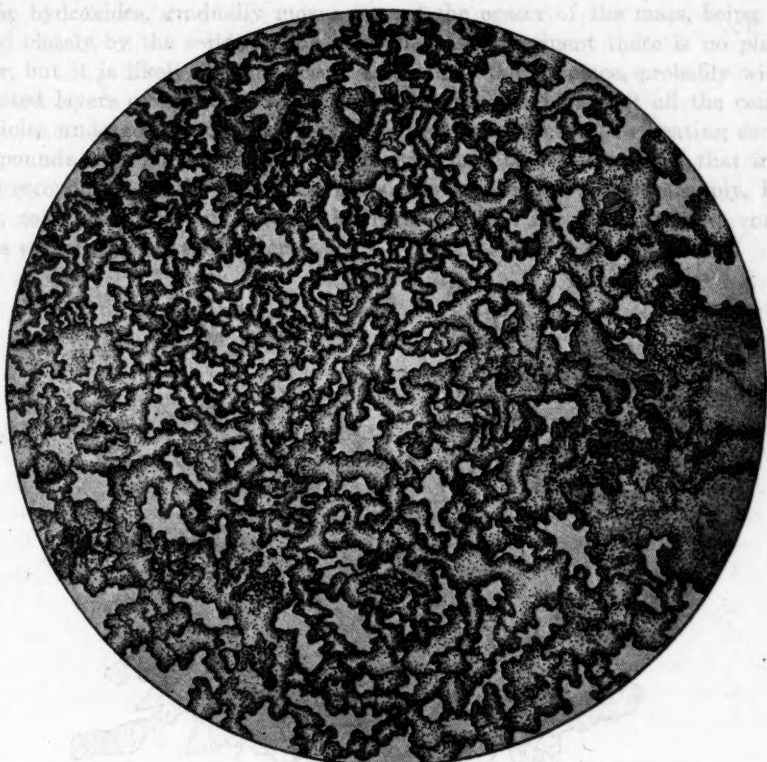


FIG. 3.—STRUCTURAL FORMATION OF PORTLAND CEMENT REPRESENTING A FIELD 0.25 MILLIMETER IN DIAMETER.



FIG. 4.—VOLUME OF 100 GRAMMES OF CEMENT MIXED WITH VARIOUS QUANTITIES OF WATER.

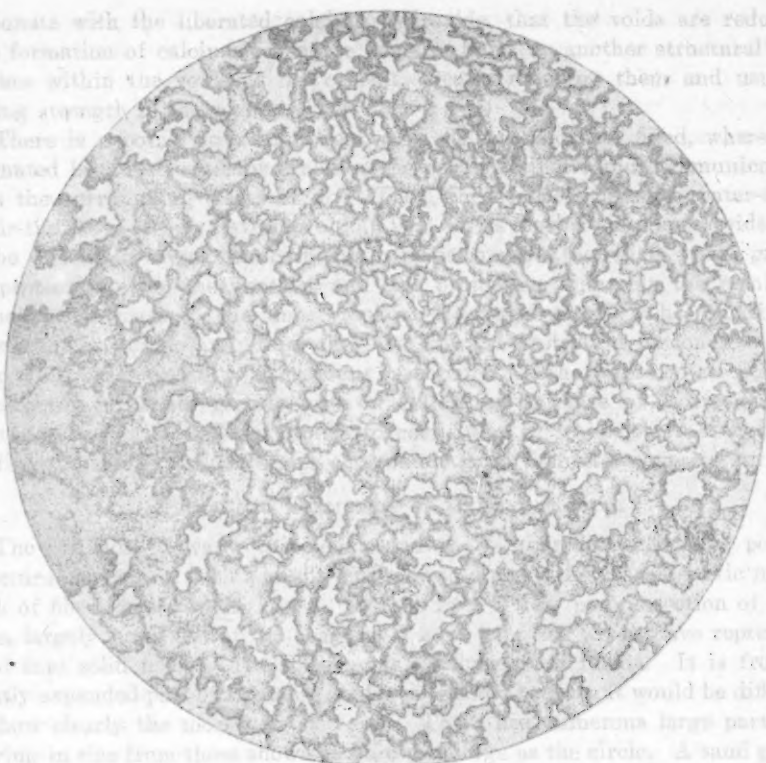


FIG. 3.—STRUCTURAL FORMATION OF PORTLAND CEMENT REGENERATION FROM A
A FORD 0.25 MILLIMETER IN DIAMETER.

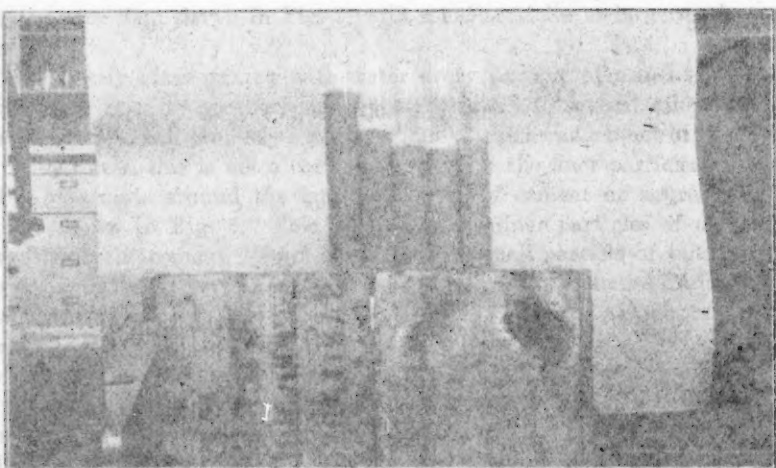


FIG. 4.—VOLUME OF 100 GRAMS OF CEMENT MIXED WITH VARIOUS
QUANTITIES OF WATER.

to the hydroxides, gradually moves toward the center of the mass, being followed closely by the solidification. In the case of cement there is no plastic layer, but it is likely that an actual solution is taking place, probably within oriented layers of water. This is not to convey the idea that all the cement particles undergo this change, for, in cement manufacture, in heating several compounds that may have different melting points, it is probable that insoluble compounds of the nature of glass may be formed. It does apply, however, to all particles undergoing hydration, which for some cements constitutes practically all the material.

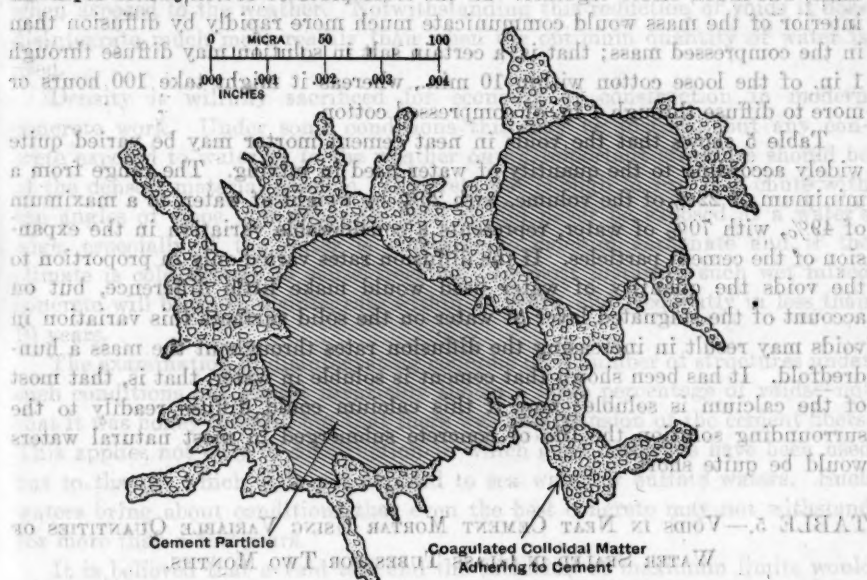


FIG. 5.—ADHERENCE OF DIRT TO PARTICLES OF CEMENT SUBMERGED IN TURBID WATER.

Percentage of voids	Water absorbed in cubic centimeters	IMPORTANCE OF POROSITY		Volume in cubic centimeters	Percentage of weight of water
		Weight after being submerged 24 hours in water	Weight before being submerged		

When good materials have been used and properly mixed, the chief factor in determining whether the concrete will withstand the weathering agents is its porosity. Air-bubbles, which under ordinary practice are not excessive, have little effect. It is the spacing between the fibers or solid particles that determines its corrosive resisting power.

This is best illustrated by comparing the volume of equal weights of cement mixed with variable quantities of water as shown in Fig. 4. In this experiment five batches, of 100 grammes each, of neat cement were mixed with different quantities of water placed in identical glass tubes. The photograph was taken one week afterward. From left to right 20, 25, 30, 40, and 60 cu. cm. of water were used. The volume is a minimum when approximately 25% by weight of water is used. This is the point of minimum voids and probably the approximate point of maximum strength. The addition of more water to a maximum of about 60 to 70% increases the voids, not by forming pockets of water, but by an expansion of the structural formation. Cement is composed

of particles varying from almost sub-microscopic size to more than 0.2 mm. in diameter. If the total mass is expanded 50%, actually it will be much more than 50% for the finer particles that fill in between the larger ones.

If a piece of loose cotton is submerged in water the volume of water around the fibers may be quite large in proportion to the actual volume of the cotton, yet the fibers may be fairly uniformly distributed throughout the mass. If this same piece of cotton were compressed to about one-tenth its original volume the fibers would still be fairly uniformly distributed throughout the mass, but much closer together. In the loose piece of cotton, water from the interior of the mass would communicate much more rapidly by diffusion than in the compressed mass; that is, a certain salt in solution may diffuse through 1 in. of the loose cotton within 10 min., whereas it might take 100 hours or more to diffuse through 1 in. of compressed cotton.

Table 5 shows that the voids in neat cement mortar may be varied quite widely according to the quantity of water used in mixing. The range from a minimum of 22% of the volume, with 20% by weight of water, to a maximum of 49%, with 70% of water, represents a considerable variation in the expansion of the cement particles. If the diffusion rates varied only in proportion to the voids the quantity of water used would make little difference, but on account of the stagnated layer of water on the solid surfaces this variation in voids may result in increasing the diffusion rates throughout the mass a hundredfold. It has been shown that cement is soluble in water, that is, that most of the calcium is soluble; now if this calcium could diffuse readily to the surrounding solution the life of concrete submerged in most natural waters would be quite short.

TABLE 5.—VOIDS IN NEAT CEMENT MORTAR USING VARIABLE QUANTITIES OF WATER SEALED IN GLASS TUBES FOR TWO MONTHS.

Percentage by weight of water.	Volume, in cubic centimeters.	Dry weight, in grammes.	Weight after being submerged 24 hours, in grammes.	Water absorbed, in cubic centimeters.	Percentage of voids.
20	30.0	60.9	67.6	6.7	22.3
25	29.0	59.5	64.2	7.7	26.5
30	25.5	47.6	54.6	7.0	27.4
35	28.0	48.3	56.7	8.4	30.0
40	21.0	34.0	41.3	7.3	34.7
45	16.5	26.4	32.2	5.8	35.2
50	16.2	23.4	29.7	6.3	38.9
55	15.5	22.0	28.1	6.1	39.4
60	20.7	28.1	36.7	8.6	41.5
70	27.0	32.8	46.0	13.2	48.9

Neglecting air-bubbles, all voids in the cement mortar were filled with water when 25% by weight of water was used, yet the mortar did not separate from the water when 60% was used. The only explanation is the force of the stagnated layers of water holding the particles apart. It is difficult to realize the force required to remove this stagnated layer from solid surfaces. A good illustration is the oil in a bearing. It may be under hundreds of pounds pres-

sure to the square inch, yet a film of oil keeps the two metal surfaces from touching. If the oil acted as a liquid next to the metal the heavy pressure would force it out.

Water is not as good a lubricant as oil but the tendency is the same. When water is added to cement to such an extent that there is an actual separation of the solids after it becomes still, the distance between the particles is so great that the stagnated layers are not joined together continuously throughout the mass, and if it were not for a decrease in voids by the formation of calcium carbonate while or after setting, this wet concrete would deteriorate rapidly when exposed to the weather. Notwithstanding this reduction of voids it does disintegrate much more readily than when the optimum quantity of water is used.

Density is wilfully sacrificed for economy of construction in modern concrete work. Under some conditions this may be justified, but any concrete exposed to water or to the weather on an important structure should be of the densest material possible. Concrete wet enough to run in a chute with the angles of slope now generally used should never be exposed at a water's edge, especially if the water is corrosive to calcium carbonate and if the climate is cold enough to cause freezing. In most instances such wet mixed concrete will begin to disintegrate within 20 years, and frequently in less than 10 years.

The examination of disintegrating concrete on a number of structures under such conditions has almost invariably shown a large percentage of voids—not that it was honey-combed, but that there was an expansion of the cement fibers. This applies not only to construction in which good materials have been used but to that in which it is not exposed to sea water or sulfate waters. Such waters bring about conditions that even the best concrete may not withstand for more than a few years.

It is believed that a void test and the inclusion of maximum limits would be desirable additions to the specifications for concrete on important exposed structures. It is realized that any addition of laboratory tests is not regarded with great favor in construction work; however, when present methods under the best of inspection do not always produce durable concrete, the small cost of such additional tests would be insignificant if they aided in producing better concrete. One procedure for making the tests that gives satisfactory results will be described.

MEASUREMENT OF MORTAR AND OF VOIDS IN IT.

Pour a sample of the concrete into a mould 4 in. or more in its least dimension, covering it to make it as nearly air-tight as possible. Allow the mixture to stand over night, or until it has hardened sufficiently so that it may be removed from the mould without breaking. Leave it for at least 24 hours in a drying oven having a temperature between 212° and 250° Fahr., then cool it to room temperature, weigh it, and submerge it in water of approximately the same temperature. Weigh it again after it has been submerged successively 5 min., 2 hours, and 4 hours. If the 4-hour weight is appreciably higher than the 2-hour, submerge it for another 2 hours, repeating until the weight is prac-

tically constant. Remove the outside water with a dry cloth each time before weighing. Measure the volume by displacement of water while the sample is wet, and, without drying, crush the sample, preferably with a hand hammer, using care not to injure the pieces of coarse aggregate. Striking the pile of crushed concrete repeatedly with a light hand hammer will crush the mortar very fine. Sift the material through a No. 8 standard sieve, and repeat the crushing and sifting until all the mortar has been removed from the aggregate. From five to ten sievings will usually be sufficient. Weigh the moist aggregate, dry it at the same temperature as was used for the concrete, and weigh it again. Determine the actual solid volume of aggregate by submerging it in water. This should be done while it is moist. Few air-bubbles within the concrete will be filled with water by submergence if the sample is not heated. It is desired to measure the voids around the cement fibers only, and no effort should be made to fill the bubble spaces with water.

The following is a very convenient form for record:

TEST FOR MORTAR AND VOIDS IN CONCRETE

Date.—October 21, 1924.

Sample.—Addition to Montebello, Filters. Settling basin floor.

Age.—1 day. Theoretical proportions in mix.—1 : 2 : 4.

Weight of dry concrete.....1288 grammes.

Submerged 5 min.....1359 grammes.....5.5% water absorbed.

Submerged 2 hours.....1390 grammes.....7.9% water absorbed.

Submerged 4 hours.....1390 grammes.....7.9% water absorbed.

Percentage of total absorption within 5 min.....70

Volume of concrete.....590 cu. cm.

Weight of wet gravel.....867 grammes.

Weight of dry gravel.....860 grammes.

Volume of loose gravel.....560 cu. cm.

Percentage of voids in loose gravel.....40.7

Actual volume of solid gravel.....332 cu. cm.

Mortar volume (590 — 332).....258 cu. cm.

Water absorption of mortar (1390 — 1288) — (867 — 860).....95 cu. cm.

Percentage of voids in mortar (95 ÷ 258).....36.8

Percentage of mortar volume to total volume.....43.7

Remarks.—Voids in mortar are excessive, caused by very wet mix.

This procedure does not give the total voids in the concrete, for there are numerous air-bubbles not displaced by the water. In fact it is believed that few air-bubbles are filled with water unless the specimen is boiled or subjected to a vacuum. Voids that might be filled so easily probably would not exist in the concrete. The test gives a fairly accurate measure of the voids surrounding the cement fibers, which is the information desired, for it is the spacing between fibers or the structural expansion of the cement that determines its power to resist the elements. Air-bubbles have little effect on the life of concrete, except possibly when they become filled with water and freeze.

This proposed test is not to supersede the slump test, but to aid in interpreting it. The aggregate has considerable influence on the slump, and, although a fairly good idea of the porosity may be obtained in this manner, it is not as accurate as the water absorption test. The requisite slump for concrete that is to be exposed to water and freezing weather is so small that con-

siderable variation in density will not greatly affect it. The void test is applicable to old concrete, but it is more difficult to remove the mortar from the coarse aggregate. Table 6 shows the voids in various concrete samples.

PROPORTIONING AGGREGATES

The writer does not believe in changing customary practice as long as the old method gives satisfactory results. The present 1:2:4 proportion of cement, fine aggregate, and coarse aggregate, especially the ratio of fine to coarse aggregate, does not give the best results over a wide range of aggregate sizes. That there is need for a more scientific proportioning of aggregates in concrete is generally recognized. Fine aggregate is quite variable, even if it is taken from the same locality, and frequently from the same source. In one locality an excellent quality of sand may be found varying in size from fine to $\frac{3}{8}$ in. in diameter, and in another locality sand and gravel may be found mixed in various proportions. The engineer is confronted with the problem of how to utilize local materials to the best advantage.

Specifications for fine aggregate usually state that it shall vary from fine to coarse, with a certain percentage passing a No. 4 sieve. The recommended specifications* of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete suggest that not less than 85% pass a No. 4 sieve. These specifications give little idea of what should pass a No. 8 or No. 16 sieve, except in an Advisory Appendix (XVI)† that recommends varying proportions of aggregates for variable sizes and slumps. It is not uncommon to find fine aggregate that will comply with these specifications with from 25 to 35% retained on a No. 8 sieve, or with more than 90% passing a No. 16 sieve. Curves of the percentages by weight of sizes determined by sieving are plotted for various sands. One characteristic of the curve is the rapid increase in size for the coarser particles. This is shown in Fig. 6, which is a plotting of several characteristic sands.

All evidence points to the fact that the effectiveness of sand for concrete work depends largely on the quantity passing a No. 16 sieve, providing it is properly graded below this size. It requires less than one-half as much sand by volume passing a No. 8 sieve for gravel that is uniformly graded from the No. 8 size to $1\frac{1}{2}$ in. in diameter. The desire to maintain the 1:2 ratio of fine to coarse aggregate probably has done more to place the dividing line between them at the No. 4 sieve size than has any scientific reason. It is not a good dividing line, and the tendency now is to establish other ratios, depending on the aggregate sizes. Since a change is essential, it is believed that much good will be done by establishing the dividing line at the No. 8 sieve size. This will be especially advantageous for the smaller jobs where the necessary fine sieves and men to operate them and calculate theoretical proportions are not available. All that will be needed is a No. 8 sieve and scales for weighing, or some means of measuring the volume. The computation necessary for proper proportioning is then simple.

* *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

† *Loc. cit.*, p. 1277.

TABLE 6.—COMPARISON OF VOIDS IN CONCRETE MORTAR.

Sample.*	Proportions in mix.	Age.	Dry weight, in grammes.	Hours submerged.	Wet weight, in grammes.	Volume of concrete, in cubic centimeters.	COARSE AGGREGATE :				MORTAR :		
							Wet weight, in grammes.	Dry weight, in grammes.	Loose volume, in cubic centimeters.	Percent- age of voids.	Actual volume, in cubic centimeters.	Total volume, in cubic centimeters.	Percent- age of voids.
1	1:2:4	1 day	1 108	4	1 177	495	746	741	470	39.3	285	210	30.5
2	"	2 days	1 386	4	1 532	682	751	746	480	40.6	285	307	33.0
3	"	1 day	1 288	4	1 390	590	567	560	560	40.7	332	238	36.8
4	"	"	1 736	4	1 777	332	512	507	330	39.1	195	137	26.3
5	"	"	766	4	802	348	492	488	330	42.2	185	163	19.6
6	"	6 months	800	24	885	338	530	515	385	38.2	207	146	41.3
7	"	1 day	1 584	4	1 708	885	1 097	1 059	680	36.0	435	315	36.8
8	"	22 days	1 231	4	1 300	533	845	839	525	38.5	335	278	32.0
9	"	1 day	1 456	4	1 596	657	933	925	630	38.8	379	278	25.9
10	"	24 days	1 180	4	1 235	515	833	816	324	38.1	324	191	25.1
11	"	8 months	491	24	537	209	335	330	200	36.5	127	83	37.8
12	"	1 day	1 450	4	1 547	632	1 034	1 027	680	37.8	423	289	37.6
13	"	11 years	444	24	461	186	286	233	165	40.0	90	87	16.1
14	1:2	10 "	577	24	602	253	475	473	310	41.6	181	55	12.0
15	"	10 "	124	24	135	55	475	473	310	41.6	181	55	20.0
16	3:4	11 "	766	24	806	349	1 183	1 133	432	35.2	352	167	22.1
17	"	6 "	725	24	824	784	1 183	1 133	432	35.2	352	167	22.1
18	"	11 "	935	24	1 025	480	513	507	330	39.1	195	137	26.3
19	"	9 months	110	24	1 135	70	53	53	20	46.0	50	20	46.0
20	1:2:4	12 "	1 246	24	1 320	535	811	806	310	30.7	235	235	30.7

*Samples 1, 4, 5, 8, 9 and 10 were mixed in the laboratory, the mixes varying from fairly dry for Sample 1 to wet for Sample 10. All other samples were from concrete on important work. Sample 27 is given to show what sometimes happens to concrete on an important structure. Samples 4, 9, and 10 were mixed with about the right proportion of water for proper handling in forms, but not wet enough to run in a flat sloping chute.

To establish this size as the dividing line between aggregates will not confuse those who wish to use the proportions recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, or the Abrams' fineness modulus. If all material passing the No. 8 sieve is classified as fine aggregate and all that is retained on the sieve as coarse aggregate, and if the proportion of fine to coarse is from 2 to 5% more than the voids in the loose coarse aggregate, the variation from the fineness modulus as recommended by Abrams* will be little for the materials ordinarily used. This is not suggested to take the place of the more accurate methods of proportioning on large jobs, or where skilled supervision is available, but to urge the need of a change in the dividing line between aggregates. The No. 8 sieve is practically an essential in the proposed void test; it more nearly coincides with the abrupt change in aggregate sizes; and it is the limit of size usually specified for brickwork and plaster mortar.

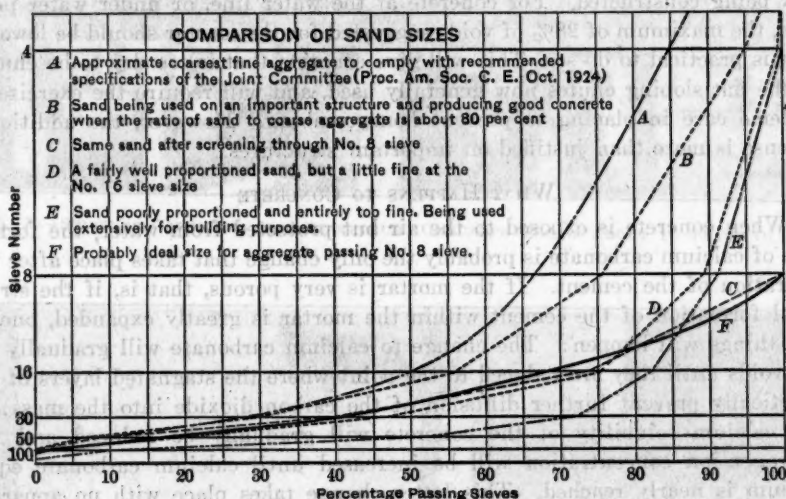


FIG. 6.

Accordingly, the following is suggested as a desirable addition to the customary specifications for 1 : 2 : 4 concrete:

The ratio of cement to aggregate shall be.....(this will depend on the character of work. The ratio, 1 : 6, is a good proportion) by volume, the fine and coarse aggregate being measured separately. All aggregate passing a No. 8 standard sieve shall be classified as fine aggregate, and all retained on this sieve as coarse aggregate. The coarse aggregate shall be graded from..... (insert largest size desired) to the No. 8 sieve size. Not more than 15% by weight shall pass a No. 4 sieve. The fine aggregate shall be graded from the No. 8 sieve size to the finest, with not more than (about 20) per cent. and not less than..... (from 5 to 10) per cent. passing a No. 50 sieve. The proportion of fine aggregate to coarse will be computed on the basis of the voids in the loose coarse aggregate. The percentage of voids plus.....(this should vary from about 2 for very coarse aggregate to about 10 for $\frac{1}{4}$ -in. aggregate; 4 is

* "Design of Concrete Mixtures," Bulletin 1, Structural Materials Research Laboratory, Lewis Inst.

about the proper figure for 1-in. aggregate) will be the ratio of the loose fine aggregate to the loose coarse aggregate. In case the specified proportions do not give a mortar volume in the concrete at least 2% in excess of the voids in the coarse aggregate the proportions shall be changed to give at least this amount. The voids in the mortar of the concrete 1 day old shall not exceed 28% of the volume of the mortar.

It is difficult to state the maximum allowable porosity for the mortar in concrete. Certainly, when it may be exposed to water and freezing weather the percentage of voids should not amount to 35 or 40 as is now frequently the case. This may be suitable for protected buildings where great strength is not necessary, but it is not suitable for outside exposures. It is believed that 25% of voids is the minimum practical quantity in cement mortar as it is first poured, except in the top finish of sidewalks and floors, where the excess water is allowed to escape and the mortar afterward is worked together to its maximum density. The treatment results in the most durable concrete now being constructed. For concrete at the water line, or under water pressure, the maximum of 28% of voids suggested for the mortar should be lowered if it is practical to do so. This will give concrete that is too dry to be chuted in the flat sloping chutes now generally used, and will require the exercise of extreme care in placing to prevent honeycombing. However, the additional expense is more than justified on important structures.

WHAT HAPPENS TO CONCRETE

When concrete is exposed to the air but protected from water, the formation of calcium carbonate is probably the only change that takes place after the hydration of the cement. If the mortar is very porous, that is, if the structural formation of the cement within the mortar is greatly expanded, one of two things will happen: The change to calcium carbonate will gradually fill the voids until they are reduced to the point where the stagnated layers of air practically prevent further diffusion of the carbon dioxide into the mass; or the calcium solubility of the concrete will gradually be reduced and the hydrogen-ion concentration will be increased until calcium carbonate equilibrium is nearly reached. This latter change takes place with no apparent weakening of the strength of the concrete; in fact, there is usually an actual increase in strength.

The only danger where the structure is protected from the weather is the possibility of the steel reinforcement rusting and breaking the concrete as shown in Fig. 7. This construction was about 10 years old and was protected from the weather, but was on the inside of a building where the air was moist most of the time. There was a layer of concrete, 1½ in. thick, over the steel. The concrete that flaked off was tested by pulverizing it and submerging it in distilled water in a closely stoppered flask for 20 days. The solution at the end of this time had a phenolphthalein alkalinity of 28 and methyl orange alkalinity of 52. These figures are considerably below the point necessary to prevent rusting of iron. The rate of penetration of carbonic acid will be slowed up considerably as the depth increases, but it may eventually penetrate the entire depth of the girder. The materials used in construction probably were good, but mixed too wet.

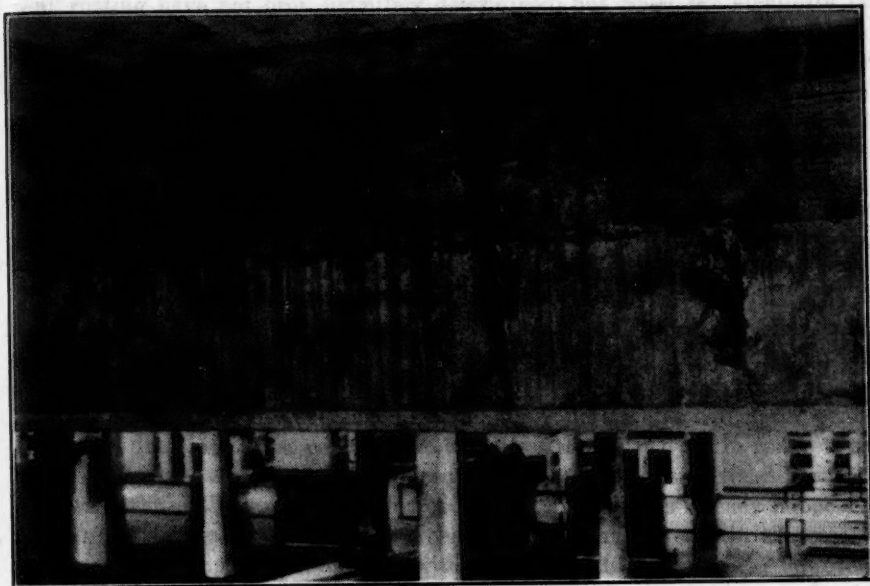


FIG. 7.—STIRRUPS CORRODING IN A REINFORCED CONCRETE GIRDER ABOUT 10 YEARS OLD.

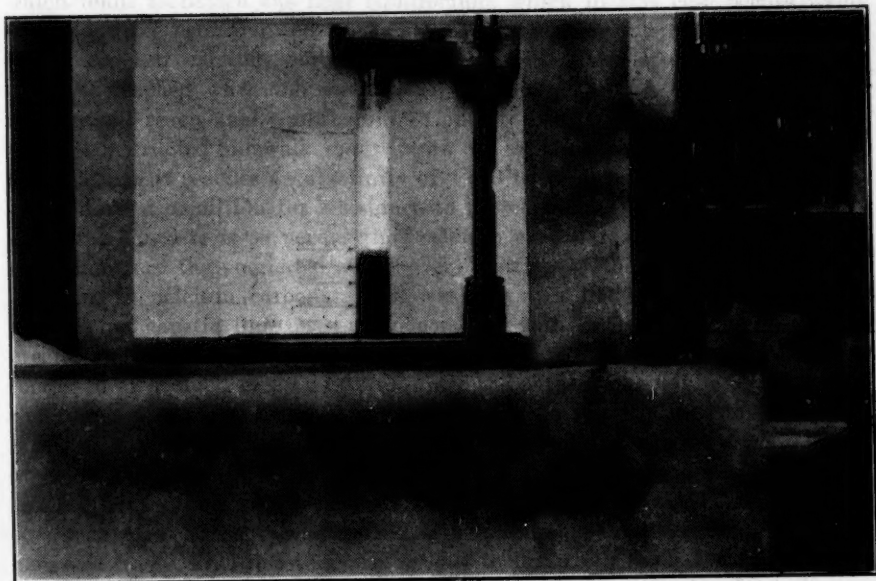


FIG. 8.—SLOW DIFFUSION RATES OF CALCIUM HYDROXIDE THROUGH ALUMINA HYDROXIDE.



FIG. 7.—STAINLESS STEEL REINFORCED BY A HIGH-TENSILE TENSILE RODS ABOUT TO TAKE OFF

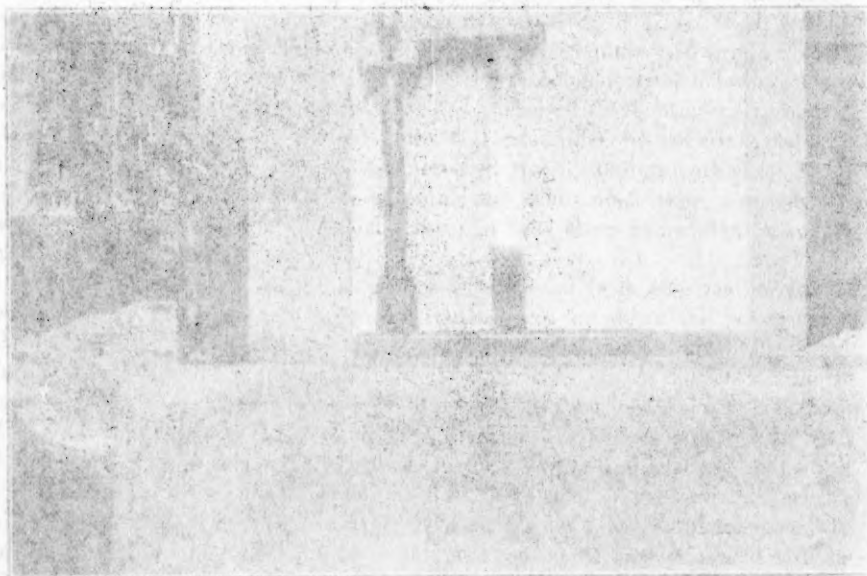


FIG. 8.—STAINLESS STEEL REINFORCED BY A HIGH-TENSILE TENSILE RODS ABOUT TO TAKE OFF

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The hydrogen-ion concentration, pH , and the alkalinity necessary to prevent rusting have not been definitely determined, but when the alkalinity is about 200 parts per million the pH should probably be as high as 11.0. Some wet mixed concrete is so porous that the CO_2 penetration will eventually reduce the alkalinity and pH below these points for some distance from the surface if not throughout the entire mass. Iron rust, however, cannot progress without space for expansion. If the expansive force is not sufficient to crack the concrete, rusting will not seriously injure the steel, but it does leave the concrete under a strain, which, combined with other stresses, might eventually cause failure. The hydrogen-ion concentration and alkalinity of a solid mass are virtually the same as the water within the pores if any is present.

If concrete is very dense the pores are filled practically to the point of preventing further entrance of CO_2 gas before much of the calcium has been changed to calcium carbonate. No rate of penetration for CO_2 can be set, for it will depend on a number of variables, the most important of which is the structural expansion of the cement within the concrete. It will never penetrate very deep into dense concrete, probably less than $\frac{1}{2}$ in., but in concrete so porous that the voids are not reduced to the stagnated stage the penetration will eventually reach the center of the mass. This has already happened in many instances, and there will probably be a great increase in the cracks produced by rusting reinforcement within the next 10 to 20 years; however, the pressure necessary to crack the concrete may give protection to deeply embedded steel.

Concrete exposed to the weather is subject to alternate wetting and drying, which tends to hasten the final equilibrium, which in this case occurs when the voids have been reduced to the stagnated stage, or when the concrete has nearly reached calcium carbonate equilibrium. This latter condition will never be reached when the concrete is very dense; only a thin layer on the outside will reach final equilibrium. Concrete $\frac{3}{4}$ to $\frac{1}{2}$ in. from the surface of a well constructed sidewalk many years old will give up calcium when submerged until it reaches an alkalinity of 1 600 to 1 800 parts per million. This is the desired condition for resistance to the elements.

If the concrete is so porous that calcium carbonate equilibrium is nearly reached before the stagnated stage, every wetting of the concrete with water corrosive to calcium carbonate will wash away a little of the calcium, for there is no caustic lime present to combine with carbon dioxide forming a precipitate. Gradually the pores will be increased. Diffusion rates will be increased at a much greater rate than are the pores. The final stage for such concrete is disintegration, which will be greatly hastened by exposure to freezing weather. Sometimes concrete that has withstood the weather for a number of years suddenly begins to disintegrate. Such cases, where good materials have been used and where the results are not affected by certain neutral salts or where there is no concentration of other detrimental compounds due to moisture travel, are almost invariably due to failure in reduction of the voids to the stagnated stage. There is probably a gradual weakening beginning a few years after construction, but it requires years to reach the point of disintegration.

In concrete submerged in water corrosive to calcium carbonate the porosity of the cement is again the main factor affecting corrosion. From a fresh exposed surface, calcium hydroxide is set free in the water. If there is considerable CO_2 , free or half-bound, in the water, calcium carbonate forms and precipitates with much of it adhering to the concrete surface. This soon reduces the outside pores to the stagnated state and prevents rapid diffusion of the calcium from the interior of the concrete to the surface. The water adjoining the surface then approaches its original *pH* and alkalinity, which has been assumed to be corrosive. In this manner there is a gradual diffusion of calcium hydroxide to the surface, a change to calcium carbonate, and a gradual dissolving of the calcium carbonate.

When this action has progressed a short distance into the concrete, probably less than 0.001 in., another force is brought into play. The dissolving of the calcium from the aluminum compounds leaves aluminum hydroxide, which builds up a gelatinous structure around the outside surfaces, tending to close the voids, and make the mass more impervious. Gelatinous silica also aids, but there is no definite proof that silica goes into solution when most of the calcium is dissolved.

These gelatinous compounds, even if apparently loosely formed, greatly reduce the diffusion rates. Some idea of the slowness of these rates may be obtained from Fig. 8. In this experiment about $\frac{1}{2}$ gramme of hydrated lime was placed in the bottom of the tube and aluminum hydroxide coagulation was added. The hydroxide was obtained by precipitation from aluminum sulfate and was washed nearly free of sulfates before being used. A small quantity of phenolphthalein solution had been added previously. The diffusion of the calcium hydroxide could be followed quite readily by the change to a pink color, as shown in Fig. 8. It required 30 days to penetrate the entire depth of about $2\frac{1}{2}$ in. Had the aluminum hydroxide been more compact the rate of penetration would have been much slower.

Unless the surrounding water is considerably below calcium carbonate equilibrium and there are no other forces aiding, such as freezing or detrimental soluble compounds, corrosion will be slow. The free access of water into the interior of the concrete, by cracks or honeycombed concrete, greatly aids in extending the corrosion into the interior.

DIFFUSION RATES AND THE CONCENTRATION OF DESTRUCTIVE COMPOUNDS WITHIN CONCRETE

The rate of seepage through concrete due to capillary attraction is usually many times the diffusion rates of most fairly soluble compounds ordinarily found in water or concrete. The finer the pores within a solid the slower are the rates with which soluble compounds within the solid will diffuse to the surface, and also the slower are the rates of water travel by capillary attraction. Evaporation of moisture from a concrete surface near the point where water is in contact with the mass, usually causes water travel within the concrete at a greater rate than the diffusion rates backwards for many of the fairly soluble compounds. For a concrete wall partly submerged, as shown in Fig. 9, the surface above the water line gives off moisture and forces by

capillary attraction water into the concrete below the water line. This not only concentrates the soluble salts originally in the water, where the moisture is evaporating, but also dissolves soluble compounds from the cement near the point where it is entering and concentrates them near the evaporating point.

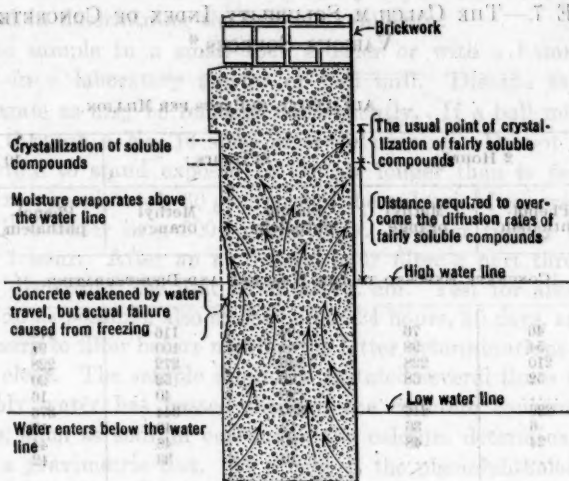


FIG. 9.—WATER TRAVEL IN CONCRETE WALL PARTLY SUBMERGED.

Soluble compounds, such as sodium sulfate, sodium chloride, sodium carbonate, magnesium sulfate, magnesium chloride, and others, may be concentrated near the surface where the moisture is evaporating, unless removed by rains or frequent washing of the surface. Neutral salts, such as sodium chloride and sodium sulfate, have the power of base exchange when the concrete has reached certain stages; that is, the calcium combined with or adsorbed by the silica and possibly the alumina is exchanged for sodium. This exchange frequently takes place when the quantity of salt in solution is at low concentration, probably less than 25 parts per million, and when the calcium in the silica and alumina has been reduced to a certain stage. Under such conditions the compound concentrating near the surface where evaporation is taking place may be sodium carbonate. Moisture evaporates from the surface of sodium carbonate and sodium sulfate nearly as readily as from a water surface, consequently, the concentration of these salts to the crystallizing point probably does not greatly reduce the evaporation rate as in the case of calcium carbonate and probably of some of the other compounds. Some salts, when confined, crystallize under pressure and may induce failure by causing the concrete to swell and crack. Actual failure is usually greatly aided by freezing weather.

CALCIUM SOLUBILITY INDEX

A number of disintegrating structures have been visited during the course of this investigation and many samples have been collected. It has been found that the quantity of calcium going into solution gives a good idea of

what has been taking place within the concrete. This is shown in Table 7. Neat cement mortar or concrete, sealed in air-tight containers immediately after mixing with water and allowed to stand for a number of years, will, when pulverized and submerged in water, give up considerable calcium. When

TABLE 7.—THE CALCIUM SOLUBILITY INDEX OF CONCRETE FROM VARIOUS SOURCES.*

Sample.	ALKALINITY, IN PARTS PER MILLION.					
	2 Hours:		24 Hours:		10 Days:	
	Phenolphthalein	Methyl orange.	Phenolphthalein	Methyl orange.	Phenolphthalein	Methyl orange.
CONCRETE EXPOSED TO THE WEATHER AND DISINTEGRATING.						
3	46	76	94	116	174	196
4	54	98	64	100	98	128
12	210	228	246	272	228	256
14	52	68	28	52	20	40
15	30	40	16	36
16	226	246	330	344	376	394
17	6	36	12	40	18	40
18	24	48	30	46	44	58
27	10	38	12	32
CONCRETE EXPOSED TO THE WEATHER AND IN GOOD CONDITION.						
2	418	448	1066	1100	1838	1874
13	450	480	890	922	1800	1850
23	1150	1194	1440	1476	1840	1878
24	1364	1400	1466	1500	1822	1850
29	524	568	908	940	1218	1244
30	800	336	360	394	410	438
SURFACE OF CONCRETE EXPOSED TO WATER CORROSIVE TO CALCIUM CARBONATE.						
1	28	60	32	48	36	46
19	0	40	0	50	0	56
31	24	52	26	54	30	50
32	32	48	28	56	28	58
CONCRETE INSIDE OF BUILDINGS AND NOT EXPOSED TO WATER.						
6	12	52	32	52
10	1 170	1 222	1 252	1 302	672	760
20	4	40	8	42	16	44
21	124	228	180	288	432	548
22	546	582	720	760	640	684
25	52	60	60	90	130	156
26	560	600	588	640	720	738
33	1 150	1 190	1 466	1 480	1 840	1 862

* All concrete except Sample 4 is more than 9 years old. Samples 2 and 13 were not crushed very fine.

the ratio of the weight of concrete to that of water is fairly large, calcium will go into solution usually to the saturation point of calcium hydroxide. Occasionally, the total alkalinity will exceed the saturation point, but this is due to the presence of alkalis more soluble than calcium hydroxide, or to certain

neutral salts. Tests indicate that, unless free lime is present, calcium saturation is never reached. By submerging pulverized samples in well stoppered containers and agitating them frequently, the saturation or equilibrium point appears to be reached within less than thirty days. The following procedure has been used in determining the calcium solubility:

Crush the sample in a small rock crusher or with a hammer, and then pulverize it in a laboratory mortar or ball mill. Discard as much of the coarse aggregate as may be removed conveniently. If a ball mill is used, sift the product through a No. 16 sieve after pulverizing. Do not allow the pulverized concrete to stand exposed to the air longer than is necessary before sealing. Mix approximately 60 grammes of the pulverized concrete with 300 cu. cm. of distilled water in a 300-cu. cm. flask. Stopper tightly and shake frequently for 1 hour. After an additional hour filter a part through a washed filter paper, discarding the first 20 or 25 cu. cm. Test for alkalinity as soon after filtration as possible, also at the end of 24 hours, 10 days, and 30 days. It is not necessary to filter before making the latter determinations if the solution is perfectly clear. The sample should be agitated several times daily.

If possibly water has passed through the concrete concentrating certain soluble salts, such as sodium carbonate, the calcium determination should be checked by a gravimetric test. As a rule, if the phenolphthalein alkalinity is only slightly less than the methyl orange alkalinity the alkali will be in the form of calcium hydroxide.

LAITANCE

Laitance is composed of a greatly expanded and partly coagulated formation of the finer particles of cement, or of any other finely divided material that may be present. Frequently, it contains numerous fine air-bubbles, but not necessarily. The voids are usually from 60 to 75% of the mass. The extremely fine particles are expanded to the limit of the stagnated stage. The mass also possesses free waterways. The communication with the outside air or water is so rapid, that the concrete is reduced to approximate calcium carbonate equilibrium before hydration is complete. If laitance is placed in a sealed container and allowed to stand indefinitely, it will maintain as high an alkalinity as good concrete, but will develop little strength because it is so greatly expanded.

It is not possible to expand cement to such a high percentage of voids because of its larger particles, but if all the cement were ground to the size of the smaller particles and sufficient water and agitation were provided, the expansion would be about as great. To a certain extent, this is what happens to the smaller particles of cement between the larger ones when an excess of water is used; that is, the smaller particles are expanded to the laitance stage. There is a possibility that a small amount of aluminum hydroxide is liberated when the water is added, and the formation of an aluminum hydroxide coagulation greatly aids in keeping the small cement and dirt particles much farther apart. Fig. 4 shows that dirt particles are readily coagulated by the calcium liberated from cement. Laitance is not cement of a different

character or composition from the remainder, but it usually contains a larger percentage of dirt that has been washed out of the aggregates.

WHAT TO DO

The investigation shows that the measurement of voids in the mortar gives an idea of what may be expected of concrete, when good materials have been used. It indicates that the voids in concrete 1 day old should not exceed about 28% of the mortar volume when the concrete is to be exposed to water or freezing weather. For concrete more than 1 year old, the voids probably should not exceed about 22 per cent. The alkalinity test, however, is the better criterion for judging old concrete.

The problem of how to take care of existing structures is a serious one. If a conduit or wall with water pressure on one side shows a leak immediate steps must be taken to stop it, for the concrete is weakened as the calcium is removed. In case of delay, until an exposed structure begins to disintegrate before making an effort to prevent it, it may be too late. Progress in making durable concrete has resulted from using good materials and exercising care in mixing and placing; yet with all these precautions many structures begin to disintegrate after a few years of exposure. Many of these structures become strong enough a few months after construction for the load they are to carry, and if this strength could be maintained they would remain safe. Structures disintegrate, even when the inspection has been good and every care has been exercised to see that good materials are used, as in the cases illustrated in Figs. 10 and 11.

Probably the same materials were used in the disintegrating slab (Fig. 10) as in the surrounding ones. This poor concrete when pulverized and submerged in distilled water gave a phenolphthalein alkalinity of 20 and methyl orange alkalinity of 40 parts per million. One of the good blocks tested showed corresponding alkalinities of 1800 and 1850, respectively. Too much water in the mixing may have been responsible for the disintegration. If the surrounding blocks do not suffer any material change from their present alkalinity they will never disintegrate.

These conditions force the realization that present-day knowledge of concrete is still quite meager. If good materials were used in the bridge abutment (Fig. 11) it seems that only one thing can account for the disintegration—a great structural expansion of the cement fibers. Tests of a sample from the abutment showed a phenolphthalein alkalinity of 16 and a methyl orange alkalinity of 36. The voids in the mortar were 44 per cent. Such a greatly expanded cement structure, if not the direct cause for disintegration, certainly is a contributing cause.

The alkalinity of concrete measured $\frac{1}{4}$ in. from the surface for structures exposed to the weather or to water should not fall below about 200 parts per million. For a depth of $\frac{1}{4}$ to $\frac{1}{2}$ in. the alkalinity should not fall below about 1200. Poor sand may produce concrete which will have a high calcium solubility, but which because of its weak strength will disintegrate in freezing



FIG. 10.—CONCRETE FLOOR EXPOSED TO THE WEATHER.



FIG. 11.—BRIDGE ABUTMENT SHOWING CONSIDERABLE DISINTEGRATION.

Fig. 10. Concrete showing corrosion after exposure to weather.



Fig. 10. Concrete showing corrosion after exposure to weather.

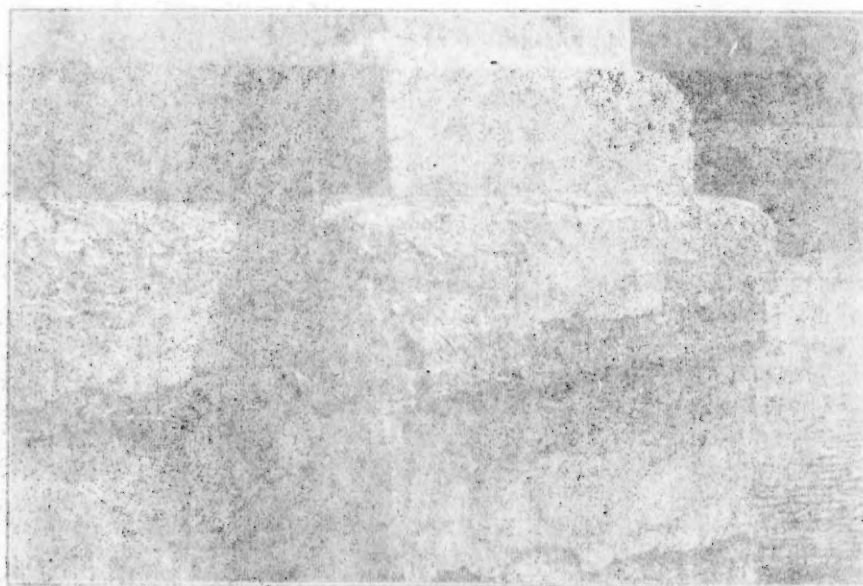


Fig. 11. Bridge abutment showing considerable deterioration.

weather. However, it is not the purpose of this paper to consider disintegration except where good materials have been used.

It seems that the most logical procedure is to examine existing structures. If the calcium solubility index at a depth of $\frac{1}{2}$ in. or more from the surface is low the concrete should be protected. It is believed that, when the safety point of alkalinity is exceeded, reliance should be placed only on some impervious surface coating, such as paint. After cracks have developed, protection is very difficult. Millions of dollars in future repairs and replacements probably could be saved by protecting exposed structures as they fall below the safety limit of calcium solubility. Many exposed structures apparently in good condition will be found to be below the safety limit, but it is likely that a gradual weakening is taking place and that eventually this may cause failure.

The writer wishes to express his appreciation of the aid given by other engineers, especially by Thaddeus Merriman, M. Am. Soc. C. E., Chief Engineer, New York City Board of Water Supply.

DETERMINATION OF THE DUTY OF WATER IN WATER RIGHT ADJUDICATIONS

The adjudication of a water right for irrigation requires by its very nature that the determination of the priority of the right, the place of use, and the quantity or extent of the right. Priority and place of use are mainly questions of fact and although important are outside the immediate interest of this discussion. The determination of the quantity of water represented by the right, the rate of diversion which may be found to represent beneficial use, or the duty of water as it is frequently called, is a matter in which engineers participating in such adjudications are directly concerned.

Under the various administrative and judicial procedures now in use in the different Western States, adjudications of water rights may be made by State administrative officers, such as the State Engineer; by the Courts, either State or Federal; or by a combination of State officers and Courts. Engineers come into contact with such determinations both from the point of view of those engaged in the conduct of the proceedings in those States where the determinations may be wholly or partly in the hands of the engineering officers of the State, or as representatives of individual claimants in all forms of adjudications. All engineers having to do with irrigation development or operation are concerned in the basis of the procedure which may be followed, and the effectiveness of the results which may be obtained as these results

* This report represents a part of the results of the work undertaken by the Duty of Water Committee of the Irrigation Division. It was presented at a conference conducted jointly by the Chairman of the Committee, S. T. Harding, M. Am. Soc. C. E., at the meeting of the Irrigation Division at Salt Lake City, Utah, July 8, 1925. The report was submitted by the Committee to the Executive Committee of the Irrigation Division. The Executive Committee has recommended its publication in proceedings in order to elicit discussion of the subject which is invited.

weather. However, it is not the purpose of this paper to consider disintegration except where good materials have been used.

It seems that the most logical procedure is to examine existing structures. If the calcium solubility index at a depth of 1 in. or more from the surface is low the concrete is probably sound.

DETERMINATION OF THE DUTY OF WATER IN WATER-RIGHT ADJUDICATIONS*

Millions of dollars in future repairs and replacements are being expended on irrigation works. It is probable that the safety of these structures is being jeopardized by the use of inferior materials and by the lack of proper maintenance.

REPORT OF THE DUTY OF WATER COMMITTEE OF THE IRRIGATION DIVISION OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

SYNOPSIS

This report discusses features of water adjudications of interest to engineers. The principles on which the duty of water is defined as indicated by Court decisions are analyzed. The character of evidence likely to be most useful in assisting the adjudicating agencies in such determinations is discussed. Illustrations of results and practices in existing decrees are included. Suggestions are made as to desirable practices in such determinations.

DETERMINATION OF THE DUTY OF WATER IN WATER-RIGHT ADJUDICATIONS

The adjudication of a water right for irrigation acquired by appropriation includes the determination of the priority of the right, the place of use, and the quantity or extent of the right. Priority and place of use are mainly questions of fact and although important are outside the immediate interest of this discussion. The determination of the quantity of water represented by the right, the rate of diversion which may be found to represent beneficial use, or the duty of water as it is frequently called, is a matter in which engineers participating in such adjudications are directly concerned.

Under the various administrative and judicial procedures now in use in the different Western States, adjudications of water rights may be made by State administrative officers, such as the State Engineer; by the Courts, either State or Federal; or by a combination of State officers and Courts. Engineers come into contact with such determinations both from the point of view of those engaged in the conduct of the proceedings in those States where the determinations may be wholly or partly in the hands of the engineering officers of the State, or as representatives of individual claimants in all forms of determinations. All engineers having to do with irrigation development or operation are concerned in the basis of the procedure which may be followed and the effectiveness of the results which may be obtained, as these results

* This report represents a part of the results of the work undertaken by the Duty of Water Committee of the Irrigation Division. It was presented, in somewhat condensed form by the Chairman of the Committee, S. T. Harding, M. Am. Soc. C. E., at the meeting of the Irrigation Division at Salt Lake City, Utah, July 9, 1925. The report was submitted by the Committee to the Executive Committee of the Irrigation Division. The Executive Committee has recommended its publication in *Proceedings* in order to elicit discussion of this subject which is invited.

determine the ability of all canals concerned to secure the water supply necessary for their successful operation with a minimum of controversy and uncertainty. Engineers are also interested directly in the procedure for the determination of the extent of the rights as it is on this question that they may most often be asked to present evidence.

The quantity of water which may represent a reasonable standard of beneficial use is a question of opinion rather than of law. The character of evidence which may influence such determinations, the standards of practice to be followed, and the terms and form of the resulting decree are all matters of engineering interest.

The following discussion attempts to analyze the present practices in such duty-of-water determinations. It is based on the results of an inquiry in which data were secured on the decrees now in effect on about thirty-five streams and also on a study of Court decisions in an effort to secure information on the principles followed by the Courts. Opinions were sought from those responsible for the administration of present decrees on the experience secured from their operation. The Committee is under grateful obligation to the large number of those who have assisted it in the collection of these data.

STANDARDS OF PRACTICE USED IN DETERMINATIONS OF BENEFICIAL USE

There are different standards that may be adopted to define the extent of use that is considered beneficial. Among these are:

- (1) Claims in original notice or in applications;
- (2) Capacity of constructed works; and
- (3) Requirements of lands served.

(1).—*Claims in Original Notice or in Applications.*—Appropriation rights were initiated in the earlier periods of development of the Western States by posting a notice of the appropriators' intention at the point of diversion and recording the notice in some county office. This has been changed in most States so that a formal application must be made to some State officer who has a varying amount of jurisdiction over the application during the period of completion. Wyoming adopted a centralized system of application in 1890 and all the remaining States in which irrigation is of importance have since adopted similar systems except Montana which still follows the former method. The claims to water rights of a great many systems now in use were initiated by the posting of such a notice of intention.

Such notices were required to include a statement of the quantity of water claimed. As there were no limitations on the extent of such claims, it was natural that the appropriators should state a quantity amply liberal to cover any probable needs. This practice is so well recognized that little weight would ordinarily be given to the quantity claimed in the notice. The notices are useful as evidence of the date of initiation, of the general character of the works contemplated, and of the location of the point of diversion. (*Hufford v. Dye*, 121 Pac. 400 (California, 1912).)

(2).—*Capacity of Constructed Works.*—The capacity of the constructed works is a more definite guide to the quantity required than the claim in the

notice as it represents the extent to which the appropriator was willing to incur costs in providing for his needs. However, as many early diversions were constructed with less definite understanding of the carrying capacities of canals or the water requirements of lands than is available at present, the carrying capacity will not usually be accepted as controlling against evidence of the actual requirements. If other standards result in a larger use than the capacity of the works, the capacity will usually define the limit of the right. (*Conrow v. Huffine*, 138 Pac. 1094 (Montana, 1914); *Nichols v. Hufford*, 133 Pac. 1084 (Wyoming, 1913); *Felsenthal v. Warring*, 180 Pac. 67 (California, 1919); *Northern California Power Co. v. Flood*, 199 Pac. 315 (California, 1921); *Hough v. Porter*, 98 Pac. 1085 (Oregon, 1909).)

(3).—*Requirements of Lands Served*.—The requirements of the lands served represent the usual present standard by which an appropriation right for irrigation is defined. There is room for much difference of opinion in regard to the character of practice that should be used in defining such requirements and also in expressing any standard in numerical terms for any specific diversion. Among the standards which may be used are the following:

- Rates of use specified by State statutes;
- Earlier local or individual practice; and
- Present practice.

The terms used are necessarily general.

Statutory Limitations.—Several States have statutes specifying the maximum rate of use which will be considered beneficial. Wyoming was the first State to adopt such a provision, diversions being limited to not to exceed 1 sec-ft. for each 70 acres irrigated. Other States have similar statutes with the specified rates varying from 50 to 100 acres served per second-foot diverted. Some States, in addition, specify the maximum number of acre-feet per acre per season. In some States in which there are no statutes on this point, a similar result is secured by the regulations of the State office having supervision of appropriation.

These provisions are an expression of legislative opinion regarding the rates of use which, in general, would be beneficial. They do not prevent a claimant from receiving larger quantities if he can present proof of a need for such larger use and there are a number of cases in which quantities in excess of such statutory limitations have been allowed. In fixing a rate of use to apply to a State as a whole, the Legislature would naturally select one sufficiently liberal to cover all usual conditions. This rate would exceed the quantity needed under more favorable conditions and a smaller rate of use would be justified in such cases. (*Joyce v. Rubin*, 130 Pac. 793 (Idaho, 1913); *Conrow v. Huffine*, 138 Pac. 1094 (Montana, 1914); *Nichols v. Hufford*, 133 Pac. 1084 (Wyoming, 1913); *Hedges v. Riddle*, 146 Pac. 99 (Oregon, 1915).)

Earlier Local or Individual Practice.—To use past practice as the basis of the extent of a right is practically equivalent, for developed systems, to accepting the constructed capacity as measuring the requirements. It is well recognized that in the earlier periods of development the use of water is less economical than in later periods. Although such excessive use may have been

permitted in such earlier periods, the Courts do not consider themselves bound to define the rights on the basis of past use where wastefulness can be shown. This principle appears to be generally accepted by the Courts although there may be much difference of opinion in individual cases as to what constitutes a sufficient showing of wasteful use to justify a decree less than former diversions. The lay and expert views on waste frequently differ.

It appears to be equally accepted that each claimant is entitled to have his right defined on the basis of the reasonable needs for the character of practice under which the right was acquired, provided such practice is within the limits of beneficial use. An appropriator irrigating wild hay may be limited to a proper use for such a crop, but will not be limited to the quantities required for other crops of less requirements because other claimants are irrigating such crops.

Cases supporting these conclusions are, as follows: *Hufford v. Dye*, 121 Pac. 400 (California, 1912); *Calif. Pastoral & Agricultural Co. v. Madera Canal & Irrigation Co.*, 138 Pac. 718 (California, 1914); *Union Colonization Co. v. Madera Canal & Irrigation Co.*, 178 Pac. 957 (California, 1919); *Felsenthal v. Warring*, 180 Pac. 67 (California, 1919); *Stinson Canal & Irrigation Co. v. Lemoore Canal & Irrigation Co.*, 188 Pac. 77 (California, 1919); *Northern California Power Co. v. Flood*, 199 Pac. 315 (California, 1921); *Oliver v. Robnett*, 210 Pac. 408 (California, 1922); *Farmers Co-operative Ditch Co. v. Riverside Irrigation District*, 102 Pac. 481 (Idaho 1909); *Washington State Sugar Co. v. Goodrich*, 147 Pac. 1073 (Idaho, 1915); *Conrow v. Huffine*, 138 Pac. 1094 (Montana, 1914); *Rodgers v. Pitt*, 89 Fed. 420 (Nevada, 1904); *Hough v. Porter*, 98 Pac. 1085 (Oregon, 1909); *Little Walla Irrigation Union v. Finis Irrigation Co.*, 124 Pac. 666 (Oregon, 1912); *in re Willow Creek*, 144 Pac. 505 (Oregon, 1914); *Salt Lake City v. Gardner*, 114 Pac. 147 (Utah, 1911); *U. S. v. Bennett*, 207 Fed. 524 (Washington, 1913); and *Nichols v. Hufford*, 133 Pac. 1084 (Wyoming, 1913).

Present Practice.—Many decisions contain such phrases as "an amount actually necessary for his use"; "a sufficient amount of water to irrigate the land in a proper manner"; "the highest and greatest possible duty from the waters of the State"; "the largest duty and the greatest use"; "the amount actually necessary"; "the inquiry was therefore not what he had used, but how much was actually necessary"; "we can require them only to use the water economically and reduce the quantity to a minimum by reasonable and cheap methods according to their situation and condition"; and, "reference should always be had to lands that have been prepared and reduced to a reasonably good condition for irrigation."

It is difficult to express in definite terms the standard of practice which is represented by a decree and the following conclusions are necessarily general. What may be termed present local use or practice appears to have been the factor of greatest influence. This does not mean that each canal will be decreed a quantity based entirely on its present practice, but that considering all canals concerned the adjudicating bodies are unwilling to fix the decrees on any basis that will require material changes in present methods. Where

variations in use by different ditches serving similar areas are shown, those using larger quantities may be limited to the quantities used by others.

Actual practice on each canal may be guiding unless evidence can be presented which demonstrates that better results are actually being secured voluntarily on some canals within the area. The standards of practice used in decrees are usually more liberal in the quantities allowed than would be required in order to make the canal systems feasible. A new project in the same area may be feasible with a water supply smaller than that to which the decrees may limit the older canals. Users under new canals may be willing to practice methods of irrigation in order to use successfully much smaller supplies than the Courts are willing to require of those who have developed their systems under conditions of more liberal supply and use. Of the decrees on which data have been obtained no instance was found in which such decrees are based on a higher standard than would be regarded as reasonable for present local practice. The decrees appear to follow rather than to lead in improvements in irrigation practice and the quantities decreed appear to have been reduced from those previously used only where strong proof could be submitted to show that many irrigators have adopted such improvements voluntarily and that the resulting practice has come to represent a reasonable standard for all users. This position on the part of the determining bodies is a natural one as the burden of proof of the reasonableness of any reduction in present practice should rest on those who would benefit by such reduction. The effort to secure a higher standard of practice is usually made by the later priorities who would benefit by such a standard.

Among decisions including such principles are the following: *State of Colorado v. State of Wyoming*, 42 Sup. Ct. 552; *Union Colonization Co. v. Madera Canal & Irrigation Co.*, 178 Pac. 957 (California, 1919); *Farmers Co-operative Co. v. Riverside Irrigation District*, 102 Pac. 481 (Idaho, 1909); *Washington State Sugar Co. v. Goodrich*, 147 Pac. 1073 (Idaho, 1915); *Beasley v. Engstrom*, 168 Pac. 1145 (Idaho, 1917); *Vineyard Land & Stock Co. v. Twin Falls Salmon River Land & Water Co.*, 245 Fed. 22 (Idaho, 1917); *Muir v. Allison*, 191 Pac. 206 (Idaho, 1920); *Conrow v. Huffine*, 138 Pac. 1094 (Montana, 1914); *Rodgers v. Pitt*, 129 Fed. 932 (Nevada, 1904); *Hough v. Porter*, 98 Pac. 1085 (Oregon, 1909); *Little Walla Irrigation Union v. Finis Irrigation Co.*, 124 Pac. 666 (Oregon, 1912); *in re Willow Creek*, 144 Pac. 505 (Oregon, 1914); *Salt Lake City v. Gardner*, 114 Pac. 147 (Utah, 1911); and *U. S. v. Bennett*, 207 Fed. 524 (Washington, 1913).

FACTORS AFFECTING THE DUTY OF WATER

The more important factors affecting the duty of water are the soil, character of crops, climate, preparation of land, cost of water, and the skill and attention used by those actually applying the water. Of these, climate and character of crops, although varying over large areas, are usually fairly constant within the areas covered by any adjudication. The preparation of land and the attention given to the application of water vary with the individual farm. The soil may vary widely within the area covered by an adjudication. It is not subject to material change and is not within the control

of the individual user. In consequence, the soil is the variable factor which is most often considered in fixing differences in rates of use within any area.

Of the adjudications on which data were secured, about one-half were based on some differences in the duty of water within the area covered and one-half made a uniform allowance. In some cases variations as large as 100% were made where conditions differed materially; the variations used were based mainly on soil conditions rather than on crops.

Soils.—The texture of the soil will materially affect irrigation practice both as to the frequency of application, the quantity applied at each irrigation, and the resulting total use. If it is conceded that the use of water does vary with the soil texture and that account of such variations should be taken in fixing the duty of water, the rates of use for different areas included in a determination will need to be variable if such soil variations occur. The soils in nearly all irrigated areas are sufficiently variable to warrant taking account of such differences.

The uncertainty regarding the practicability of making a soil classification of sufficiently definite character to justify varying the rate of use based on such a classification appears to have prevented a more complete consideration of soil texture. Although soil texture has been considered in some cases, the tendency appears to have been to allow an additional quantity of use to soils of coarse texture or to allow enough use to all soils to cover the needs of the more porous types. To reduce the rate of use on soils of more favorable texture is just as logical as to increase the use on those of less favorable character. A practical limit must be set in any effort to base rates of use on a soil classification, but even in areas of considerable mixture of soils a practical classification can be made which will be more nearly representative of actual needs than might be obtained on the basis of generalities only.

Cases in which differences in rates of use due to differences in soils have been upheld are, as follows: *Washington Stage Sugar Co. v. Goodrich*, 147 Pac. 1073 (Idaho, 1915); *Joyce v. Rubin*, 130 Pac. 793 (Idaho, 1913); *Rodgers v. Pitt*, 129 Fed. 932 (Nevada, 1904); *in re Waters of Umatilla River*, 168 Pac. 922 (Oregon, 1917).

The determinations of the Division of Water Rights of California have varied from a diversion of 1 sec.-ft. for 20 acres on Hat Creek for soils considered particularly difficult to handle to 1 sec.-ft. for 90 acres on heavy soils on Oak Creek. For coarse soils on the upper areas of Oak Creek, 1 sec.-ft. to 25 acres was decreed, the rights of all users being accepted by stipulation. On the largest stream adjudicated—Stanislaus River—the decrees for the smaller ditches serving mountain areas generally gave 1 sec.-ft. to 40 acres, whereas those of the Oakdale and the South San Joaquin Irrigation Districts in the San Joaquin Valley gave 1 sec.-ft., to 80 acres.

The Salt River decree in Arizona, in which the rights of about 4 800 owners were separately defined in terms of the irrigable area of each owner, was based on a uniform delivery equivalent to 1 sec.-ft. for 133 acres.

In Idaho, 1 miner's inch per acre was used for bench-lands and 1.1 miner's inches per acre for bottom-lands on the Boise River, and 1 to 5 miner's inches

per acre for different areas on Big Wood River. In Idaho, 50 miner's inches are equal to 1 sec-ft.

In Montana, in the West Gallatin decree, the rates varied from 0.75 miner's inch per acre on heavy black loam to 1.25 miner's inches per acre on gravelly land. On the smaller streams of the State different rates of use have not been general, many such decrees being based on 1 miner's inch per acre. In Montana, 40 miner's inches are equal to 1 sec-ft.

The decision of the Special Master of the Federal Court in the adjudication of Truckee River in Nevada (still pending) decrees different rates of use to different areas based at least partly on difference in soil conditions.

In Oregon, rates of use of 2.0 to 3.0 acre-ft. per acre were decreed for different areas on the Deschutes River and 1 sec-ft. for 40 to 80 acres, or 3 to 6 acre-ft. per acre for different soil conditions on the Umatilla River.

The quantities given apply to the amounts diverted unless otherwise stated.

Crops.—The crops within any area are subject to variation both over longer cycles of years and temporarily due to changing economic conditions. The more complete development of an area usually results in greater diversity of crops which, in turn, tends to reduce the average quantity of water used, as the crops first grown are generally ones of larger water requirement, such as forage. This is not the case, however, in areas where grain is largely grown in the earlier years. General crop changes occur gradually; temporary changes may be rapid, such as the growth in single seasons of large areas of some crop for which the prospective price is attractive. Where rights have been acquired based on the irrigation of one class of crops and it is proposed to change the crops so as to result in a material change in the time of use or an increase in the quantity used, such changes may be opposed by other users who have made appropriations based on the previously existing conditions. If the earlier practice has been well established such opposition would be expected to be successful.

Court decisions indicate that a claimant is entitled to a quantity of water sufficient to meet the needs of his lands for the type of crop which he has been accustomed to growing or which he may grow under reasonable crop rotations. Decrees for individual users or under small canals may be based on the requirement for the crop of largest use, usually forage. For larger areas, diversity of crops is to be expected and the needs can be based on the requirements for the proportion of different crops which has obtained during recent years. Although there may have been a definite tendency toward crops of smaller needs and although it may reasonably be anticipated that such tendency may continue, it is hardly to be expected that those making the decision will reduce the right below the quantity needed for the crops being grown at the time of the determination.

Courts are guided by local practice as to the crops the irrigation of which may be considered beneficial. The irrigation of native grasses is a recognized beneficial use, but it must be practiced under conditions representing proper local standards in the handling of the water. The fact that some claimants may desire to irrigate crops of smaller water requirement will not justify lim-

iting all rights to the needs of such crops where other rights have been used for crops of larger need.

In the decree on Salmon Falls Creek, in Idaho, the Federal Court based its findings on a use of 2.5 acre-ft. per acre for forage crops and 1.5 acre-ft. per acre for other crops, one-half of the area being used for each type of crop. These quantities are to be measured at the farm head-gate.

The recent determination on Humboldt River made by the State Engineer of Nevada is based on a use of 3 acre-ft. per acre for harvest crops, 1.5 acre-ft. per acre for meadow pasture, and 0.75 acre-ft. per acre for diversified pasture. (*Rodgers v. Pitt*, 129 Fed. 932 (Nevada, 1904); *U. S. v. Bennett*, 207 Fed. 524 (Washington, 1913); *in re North Powder River*, 144 Pac. 485 (Oregon, 1914); *Muir v. Allison*, 191 Pac. 206 (Idaho, 1920).)

Climate.—Climate enters into water-right determinations due to its influence on the length of the irrigation season, the crops grown, and the need for irrigation, as water used in irrigation is supplemental to that obtained from natural precipitation. There is less difference in the rate at which water will be used during the period of maximum demand than there is in the length of the irrigation season in different localities. The amount and occurrence of rainfall will influence the amount of irrigation required. Such differences in rainfall more usually affect the time of the beginning of the irrigation season. In localities where the rainfall continues into the growing season, as in the Plains Area east of the Rocky Mountains, the irrigation season may be delayed until later in the crop season than is the case where the rainfall occurs almost wholly in the winter months as in the Pacific Coast States.

Differences in use based on differences in climate were adopted on the Beaverhead River in Montana where different rates of use were decreed for the upper and lower valleys. These differences may be due to the combination of partial separate decrees rather than to a distinct difference based on climate. A decree in Apache County, Arizona, allowed 1 sec.-ft. to 90, 110, and 180 acres, respectively, for areas at different altitudes.

Preparation of Land.—The character of preparation of the land may make a great difference in the quantity of water used. Where the surface is uneven, greater average depths must be applied in order to cover the higher areas. The flooding of areas of too large size results in excess percolation loss.

It is usual to find a definite tendency toward improvement in the preparation of land as an area has been longer under irrigation. This is due both to the increased resources of the owners which enable them to incur greater costs in preparing land and to the benefit of longer experience under local conditions and the development of methods best suited to such conditions. The best support for a contention that older users should have their rights reduced to the quantity required under better methods of land preparation would appear to be a showing that such improved methods were in voluntary use in the area concerned by a sufficient proportion of the users to have demonstrated their advantages and economy. Efforts to secure decrees which would require general changes in the methods of preparing land where such support was not available have not been successful. (*Foster v. Foster*, 213 Pac. 895 (Oregon,

1923); *Farmers Co-operative Ditch Co. v. Riverside Irrigation District*, 102 Pac. 481 (Idaho, 1909); *Beasley v. Engstrom*, 168 Pac. 1145 (Idaho, 1917).)

Cost of Water.—The cost of water is usually an indirect factor. It may affect the duty of water due to its influence on the methods of preparing land and handling water. The cost which it may be feasible to incur is limited by the value of the crop returns. The rates charged for water may influence the use of water in actual practice, but do not appear to have entered directly into the determination of water rights.

Skill and Attention Used in Applying Water.—The skill or attention of the labor used in applying water is closely related to the character of land preparation. Claims for a quantity of water which will permit the land to be irrigated with only occasional attention to the water are frequently made in areas irrigating pasture or forage. Such practices are judged by their reasonableness in comparison with the general practice of the area. In many areas the cost of labor in the application of water still exceeds the cost of the water itself, so that the extent to which improvements which would increase the labor costs can be expected may be limited.

SUMMARY OF FACTORS CONSIDERED IN FIXING THE DUTY OF WATER

Although there are a number of decrees in which a uniform rate of use was decreed to all claimants, these illustrations appear to justify the conclusion that determining agencies, whether an administrative officer, such as the State Engineer, or a Court, either State or Federal, will decree varying rates of use to different claimants whenever they conclude that differences in soils, crops, or other conditions exist which warrant differences in the rate of use. Although soil conditions may vary within relatively small areas, the tendency appears to have been to distinguish mainly differences in the conditions for general classes or divisions of the area rather than for individual owners or ditches. Presumably, however, if evidence could be presented supporting such differences the determining body would feel free, as far as principles are concerned, to vary the use for smaller areas. With the increasing attention that is being given to the factors which affect the duty of water it is reasonable to expect that the adjudicating agencies will tend to give closer attention to the details of the water requirements of the lands served with the result that variations in the rate of use allowed may be more frequent in the future than in the past.

TERMS USED TO EXPRESS THE AMOUNTS WHICH ANY RIGHT MAY BE ENTITLED TO DIVERT AND THE SEASON OF DIVERSION

The older decrees were expressed almost entirely in terms of continuous flow. As storage has become of importance it has been necessary to express some decrees in terms of the total quantity per season and to limit the period of diversion for direct use. In the older decrees, shortages in supply generally occurred sufficiently late in the season to be within the period of maximum rate of use so that it was not necessary to define the beginning of the irrigation season. The periods of diversion were left to the discretion of the users, the

right permitted diversion up to its maximum rate at any time flow was available and a beneficial use could be made of the water. A reduction in diversion could be made when waste occurred.

The Salt River decree in Arizona permits diversion throughout the year as climatic conditions result in continuous use. On Hat Creek, in California, the decrees are effective from May 1 to October 27, no limitations being placed on use during the remainder of the year. On the West Fork of Carson River, in California, Big Lost River, in Idaho, Silvies and Umatilla Rivers, in Oregon, Logan River, in Utah, and West Okanogan, in Washington, diversion is limited to periods between specified dates.

The Bear River decree of the Federal Court for the Idaho District specifies a diversion season from April 20 to September 30, with the rate of diversion from April 20 to April 30 and from September 15 to September 30 limited to 40% of that allowed during the remainder of the period.

On the Boise River, the decree now in present use is not final on the duty of water, the tentative allowances being adjusted during the season as the stream flow diminishes. The following order illustrates the practice used:

"The various rights as adjudicated in the so-called Stewart Decree shall receive 100% until the natural flow of the water of Boise River shall decrease or until all the rights in said decree cannot receive 100%, at which time the various rights as adjudicated in the so-called Stewart Decree shall first be cut to 75% of the amount of water decreed by the Stewart Decree as the natural flow of Boise River decreases, beginning with the latest rights and proceeding to the earliest rights in the order fixed in said Stewart Decree, and after all rights shall have been reduced to 75% of the amount fixed in the Stewart Decree, should the natural flow of the waters in the Boise River decrease below the amount necessary to supply said 75% of the water rights as decreed in said Stewart Decree, then the various rights, beginning with the latest and proceeding to the earliest as aforesaid, shall be reduced to 60% of the amount specified in said Stewart Decree and 60% of the amount specified in said Stewart Decree is hereby fixed and determined as the highest duty of water for the year 1919."

This is equivalent to using a higher standard of practice during periods of small supply in order to reduce the hardship of such shortages on rights of later priority. This method has been in use for several years, apparently with generally satisfactory results.

On the Weber River, in Utah, the proposed determination by the State Engineer is based on the use during high-water stages of 1 sec.-ft. to 60, 65, and 70 acres for the upper, middle, and lower river areas, with all rights reduced to 1 sec.-ft. to 80 acres before any rights are cut off.

On the Big Lost River, in Idaho, a decree accepted by stipulation by all parties provides for diversion from April 20 to October 31, diversion during April and October to be at the discretion of the Water Commissioner.

On Logan River, in Utah, all the older rights were decreed a priority of May 1, 1860, and a schedule provided for stream flows of 400 sec.-ft. or less by which different rights receive somewhat different percentages of reduction as the flow diminishes. A similar general method is followed in the temporary schedule now in use on Kings River, in California, in which different rights

are entitled to divert different quantities at different stages of stream flow, the rights being listed for each variation of 100 sec.-ft. in stream flow up to a total discharge of 10 000 sec.-ft.

For the Humboldt River, in Nevada, the rates of use in acre-feet per acre for different crops, previously mentioned, are proportioned to the length of the irrigation season, harvest crops being irrigated over a 180-day season. The diversion rights are expressed in the decree in terms of second-feet.

The definition of rights in terms of acre-feet per acre per season has been more usual in the administrative determinations in Oregon. Total acre-feet per season for each right, with a limiting rate in second-feet, were used on the Deschutes, Grande Ronde, Hood, Umatilla, and Wallowa Rivers. On the Silvies River, 1.5 acre-ft. per acre per month prior to June 1 and 1 acre-ft. per month after June 1, to be taken at a rate of not to exceed 1.40 sec.-ft. per acre was used.

The following quotation from the decision in the case of the State of Wyoming v. State of Colorado (42 Sup. Ct. 568) represents a recent expression of the U. S. Supreme Court:

"A much larger amount is claimed, but our finding restricts the amount to what the evidence shows is reasonably required, which is 1 acre-ft. per acre for the larger part of the lands, 2 acre-ft. per acre for a part, and 2½ acre-ft. per acre for the remainder."

It appears to be well established that the determining bodies may define the rights in such terms as they consider to represent beneficial use; that the season may be restricted to those periods during which use is beneficial and that specific dates may be defined limiting the season of use; and that in addition to the more usual former practice of defining merely the maximum rate of diversion, the total quantity of use for the season may be defined. The limiting of the total quantity that may be diverted during the season, leaving the user to adjust the time of its diversions to his needs subject to some limitation on the maximum rate at which he may divert, seems to represent a desirable basis on those streams where storage as well as direct rights exist. Such a basis protects the prior right in an adequate supply under reasonably flexible conditions of use. It also defines the total use so that the supply available for storage can be determined. Its terms are sufficiently direct and specific to permit practical operation of the stream.

Decisions involving maximum rate of diversion with total acre-feet per acre limitation, are: *Foster v. Foster*, 213 Pac. 895 (Oregon, 1923); *in re Willow Creek*, 144 Pac. 505 (Oregon, 1914); *in re North Powder River*, 144 Pac. 485 (Oregon, 1914); *Laurence v. Brown*, 185 Pac. 761 (Oregon, 1919); and *Vineyard Land & Stock Co. v. Twin Falls Salmon River Land & Water Co.*, 245 Fed. 22 (Idaho, 1917).

Decisions involving different rates in low water period are: *In re Willow Creek*, 144 Pac. 505 (Oregon, 1914); *in re North Powder River*, 144 Pac. 485 (Oregon, 1914); and *Laurence v. Brown*, 185 Pac. 761 (Oregon, 1919).

Decisions involving season limited by fixed dates include *Foster v. Foster*, 213 Pac. 895 (Oregon, 1923).

CONVEYANCE LOSSES

In general, all decrees define the extent of the right at the point of diversion. This is essential usually for purposes of practical administration. The decree includes any losses in conveyance to the point of delivery to the land owner. In some determinations no distinction is made separately for conveyance losses. In others, the requirement at the land is determined and the diversion right based on increasing this by some specified quantity for conveyance losses. In the Salt River decree, 1% per mile of canal was used. Actual measurements of loss for each ditch have been used in some cases, such as the West Gallatin in Montana and the Deschutes and Grande Ronde in Oregon. The following provision is contained in the decree for Umatilla River, in Oregon:

"In all cases the water master, where the works are in good condition, may at his discretion allow an increased diversion for such seepage and evaporation, which increased diversion shall be determined by the water master according to the actual seepage and evaporation in the diversion works, but in no case shall such increased diversion exceed 20% of the amount allowed by this finding."

The diversion requirement can be determined best by estimating the delivery requirement and the conveyance loss separately and combining the result. The delivery requirement and the conveyance loss have no consistent relationship. The lengths of canals serving similar soils and having similar delivery requirements may vary widely so that a decree based on average diversion requirements may be unfair to canals of greater loss.

The construction and maintenance of canals must represent similar standards of reasonably good practice to that expected in the application of water to the land. Where conveyance losses are excessive, the right may be limited to a reasonable loss.

Cases covering conveyance losses include *Town of Sterling v. Pawnee Extension Ditch Co.*, 94 Pac. 339 (Colorado, 1908); *Wheat v. Cameron*, 210 Pac. 761 (Montana, 1923); *Basinger v. Taylor*, 211 Pac. 1085 (Idaho, 1923); *Clark v. Hansen*, 206 Pac. 808 (Idaho, 1922); *Santa Cruz Reservoir Co. v. Ramirez*, 141 Pac. 120 (Arizona, 1914).

ROTATION IN DELIVERY

Nearly all decrees include some diversions serving relatively small areas. If the right for such an area is expressed in terms of its average need the resulting diversion may be too small for effective use. To meet this condition, rotation in delivery is frequently practiced. Under larger canals such rotation is practiced among the different users without affecting the diversion from the stream. Under small canals, the users may desire to divert at twice the average rate for one-half the time or make such other adjustments as may meet their needs. To avoid confusion in the decrees it is usual to define each right in terms equivalent to its continuous diversion, leaving such adjustments to be handled by the one in charge of administering the stream under general clauses in the decree.

Where such conditions exist, it is usual to permit rotation subject to provisions under which later priorities may be protected in securing the supply to which they are entitled. Such provisions usually require the one seeking to divert a surplus to secure the approval of the water commissioner and to arrange an exchange with other owners desiring to rotate.

There appears to be some uncertainty as to the right of a Court to require rotation or to define each right so that rotation must be practiced in order to make irrigation feasible. The general tendency has been toward a recognition of rotation as a part of usual practice and toward holding that beneficial use may be defined on a basis which may result in requiring rotation for successful irrigation.

Rotation is permitted in the Salt River decree, in all decisions of the California Division of Water Rights, in the West Gallatin in Montana, the Big Wood River and Salmon Falls Creek in Idaho, permitted or required in various determinations of the State Engineer in Oregon, and permitted in the Bear River decree in Utah. No cases were found in which it was prohibited; the remaining streams appear to represent omission of any such provisions rather than objection to it unless the case of *Muir v. Allison* is considered as an exception to this statement. (*In re Willow Creek*, 144 Pac. 505 (Oregon, 1914); *in re North Powder River*, 144 Pac. 485 (Oregon, 1914); *Muir v. Allison*, 191 Pac. 206 (Idaho, 1920); and *Reno v. Richards*, 178 Pac. 81 (Idaho, 1919).)

CHARACTER OF EVIDENCE ON THE DUTY OF WATER WHICH APPEARED TO BE OF
MOST INFLUENCE WITH THE ADJUDICATING BODY

In the older cases little evidence of an expert character was presented. Technical evidence, such as measurements of canals, was often used, but this was generally for the purpose of basing the claims on records of past diversion rather than to determine the proper duty of water. More recent irrigation developments usually include large units which plan to utilize all or nearly all the remaining unappropriated flow of streams. The water supply available is the difference between the total run-off and the existing rights. The more closely the existing rights can be limited in their quantity, the larger is the remaining unappropriated water. Such limitations are in the public interest in permitting a larger total utilization of a given stream provided the limitations placed on the earlier rights are not unreasonable. As previously noted, the adjudicating bodies have not used such high standards in defining existing rights as to impose unreasonable limitations.

The advantage to be gained by such later rights from a restriction of the earlier users and the extent of their interest due to the size of many of these larger projects have resulted in the employment in many cases of expert witnesses to testify on the proper use of water. Such witnesses have more usually testified in favor of higher standards of practice than those advocated by the users themselves or those directly engaged in the operation of the existing systems. This has led to the general classification of such expert witnesses as high duty advocates, as contrasted with the usual farmer or direct experi-

ence advocates of larger quantities of water. Many attempts have been made to rebut such expert testimony on the ground that it was not based on sufficient familiarity with the areas involved, that it represented theoretical results rather than practical, that the costs necessary to accomplish the results recommended would be beyond the means of those now irrigating, and that prior users were legally entitled to such quantities of use as might be required under the general methods existing in the area. Rebuttal to the evidence of users in the area is usually based on the claim that the ordinary irrigator although experienced in the application of water is not qualified in water measurement so that his opinions on the quantities he had used are not competent evidence. As the numerical results presented by the two classes of witnesses are frequently relatively far apart the resulting decision may offer opportunity for at least a general judgment as to the evidence which appears to have been given weight in the decisions.

On the basis of the information assembled by the Committee, the decrees made by Courts appear to have been based largely on the testimony of those directly engaged in the use of water in the area under adjudication. Where expert witnesses have been able to qualify on the basis of both general experience and of experience in the locality, or where their opinions are supported by records of use by the better irrigators of the area or by experimental determinations under local field conditions, their conclusions appear to have been of more assistance to the case. Where the conclusions have not been supported by local data or have been based on experiments under conditions not representative of field conditions little weight appears to have been given to the evidence.

In the West Gallatin and Beaverhead Rivers in Montana expert irrigators were engaged by the Court to make determinations in the field and advise the Court in defining the water requirements. Irrigators having local experience were selected. In the adjudication of Hood River in Oregon, local trials of the use of water were made.

The water requirements depend on the number of irrigations and the quantity used at each irrigation. The one irrigating under a given set of conditions is usually fully competent to testify regarding the frequency of irrigation required although he may not be able to measure properly the quantity of water used. A logical determination would be secured by utilizing direct experience on the number of irrigations required, together with competent expert evidence on the quantity required for each irrigation, based on actual measurement under proper field conditions representing the standards of practice that the Court desired to follow in its decision. Such a basis for the delivery requirement when increased to include canal conveyance losses based on actual observations would appear to utilize both direct and expert evidence to the best advantage.

These comments apply mainly to adjudications by Courts. In the adjudications by administrative officers of the State, direct observational data are collected by the engineer conducting the adjudication. The entire procedure is in the hands of those experienced in such work under conditions

where the one making the determination is not limited to evidence brought before him, but may and usually does conduct his own investigations. For such determinations controversies between expert and lay witnesses are less usual. There have been relatively few cases in which the conclusions of such procedures have been modified on appeal to the Courts.

Among the decisions discussing the character of evidence on the duty of water are: *Stinson Canal & Irrigation Co. v. Lemoore Canal & Irrigation Co.*, 188 Pac. 77 (California, 1919); *Pabst v. Finmand*, 211 Pac. 11 (California, 1922); *Farmers Co-operative Ditch Co. v. Riverside Irrigation Dist.*, 102 Pac. 481 (Idaho, 1909); *Idaho Irrigation Co. v. Gooding*, 285 Fed. 453 (Idaho, 1922); *Rodgers v. Pitt*, 129 Fed. 932 (Nevada, 1904); *Anderson v. Bassman*, 140 Fed. 28 (Nevada, 1905); *Little Walla Irrigation Union v. Finis Irrigation Co.*, 124 Pac. 666 (Oregon, 1912); *Foster v. Foster*, 213 Pac. 895 (Oregon, 1923); *Sharp v. Whitmore*, 168 Pac. 273 (Utah, 1917); *Pasco Fruit Lands Co. v. Timmermann*, 152 Pac. 675 (Washington, 1915); and *Nichols v. Hufford*, 133 Pac. 1084 (Wyoming, 1913).

SHOULD JURISDICTION BE RETAINED TO PERMIT REVISION OF THE DUTY OF WATER?

Standards of irrigation practice change similarly to the standards of other features of crop production. Increasing land values call for increasing average yields in order to support such values. Such increases in yields require and support better methods of preparing land and more attention to the application of water. Public interest in the best utilization of the limited water resources supports a continually higher standard of practice as the basis of defining beneficial use. All these factors tend toward a reduction in the quantity of irrigation water used for each acre of land.

As a recognition of such changing conditions, and in order to permit later adjustments in decrees, it has frequently been urged that jurisdiction should be retained in water-right adjudications so that the terms of the decrees may be later modified to meet such changes. The Courts retain partial jurisdiction from year to year in order to provide for the appointment of water commissioners for the administration of the decree. Provisions for retaining jurisdiction to permit re-opening the case for further evidence on the duty of water and possible modifications in its terms have been made in the Salt River decree in Arizona, the Boise River and Salmon Falls Creek decrees in Idaho, and the Silvies River decree in Oregon. The Powder River decree in Oregon is left open for further evidence on seepage and evaporation. The determination of the State Engineer on the Humboldt River in Nevada is to be interlocutory for three years for further consideration of the duty of water. The findings of the Special Master of the Federal Court on the Truckee River in the same State propose that jurisdiction shall be retained for purposes of possible revision of the duty of water allowances.

The State Engineer of Utah has provided in the recent adjudications made by his office that the decrees may be re-opened after five years to review the duty of water findings. The Utah statutes provide for proceedings by which a re-determination of rights may be made where it can be shown that wasteful use occurs.

Although provisions for re-opening of decrees as stated were found, no case was found in which such re-opening had occurred unless the general determination on the Sevier River in Utah by the State Engineer is regarded as such a re-opening of the different old Court decrees on this stream. As none of these Court decrees included all users on Sevier River, the present determination is hardly a re-opening of procedure in which all rights have participated. In all cases those supplying information regarding these streams expressed the opinion that such a re-opening would at present be inadvisable, the reason more usually given being that the probabilities of securing a more favorable decree on the duty of water would be more than offset by the disadvantages of controversy, cost, and uncertainty in rights that would result.

The conclusion appears warranted that provisions for retaining jurisdiction so that such cases may be re-opened may be desirable on many streams, but that the probabilities of such re-opening are rather limited at least until conditions and standards have changed materially from those existing at the time of the original decree.

Decisions discussing the right to retain jurisdiction include: *Mays v. District Court*, 200 Pac. 115 (Idaho, 1921); and *Big Cottonwood Tanner Ditch Co. v. Shurtleff*, 189 Pac. 587 (Utah, 1920).

HAS THE OPERATION OF THE DECREE BEEN SATISFACTORY AND IN WHAT WAYS, IF ANY, COULD IT BE IMPROVED?

Although the comment is frequently made that methods of adjudicating water rights, particularly by Court procedure, are unsatisfactory, the replies to the questionnaire in nearly all cases were to the effect that the decrees were operating fairly satisfactorily. As the opinions secured were generally from people directly connected with the adjudication or its operation, although usually not representing any particular interest on the stream, the results may be regarded as representative of the general opinion. A noticeable feature of many determinations made by State administrative officers is the proportion of such cases in which the findings by such officers are accepted by stipulation before the decision is made. In some cases the opinion was expressed that a higher standard in defining the duty of water, particularly in the earlier decrees, would have been in the interest of the public. Greater definiteness in the terms and conditions were also mentioned as possible improvements. In general, however, it appeared to be the opinion of those replying that any such advantages that might be gained by revision would be offset by the disadvantages of delay, cost, and controversy.

CONCLUSIONS

The following general conclusions are the result of the Committee's study of principles and practice in the present procedure for defining the duty of water in water-right determinations:

- 1.—Determining agencies are guided mainly by present standards of reasonable practice and will not accept a standard of practice which is not

well supported by rather extensive actual use, adopted voluntarily by existing local canals.

2.—Those endeavoring to secure the adoption of higher standards of practice in such determinations may well be guided by the previous statement and will usually find their efforts more productive of practical results if it is observed. An endeavor to secure decrees representing a standard of practice not so supported may result in the determination being based on a less desirable standard that might have been followed if a more practical presentation of good practice had been made.

3.—The use of the direct experience of local irrigators to determine the number of irrigations required, with evidence on the proper depth per irrigation based on measurement under actual local field conditions representing the standard of practice it is desired to follow, represents the most logical method of using both direct experience and expert opinion.

4.—A form of decree which will meet the requirements for direct-flow rights on streams where storage is to be used will be one defining the maximum rate of diversion with an additional limitation on the total quantity that may be diverted during the season. This form of decree has been used sufficiently to establish its legality. The fixing of the irrigation season by specifying the dates for beginning and ending the diversion may be desirable or not depending on local conditions; where a limit is placed on the total seasonal diversion in quantitative units such dates for diversion are less essential. Rotation should be permitted and encouraged and the decree if based on a reasonable standard of use will generally result in practically requiring rotation among smaller diversions.

5.—Improvements in the standards of practice used as a basis in defining water rights are dependent mainly on the adoption in general irrigation practice of more of the present knowledge regarding the advantages of a more economical use of water. Legal definitions of water rights can be expected to follow such improvements in general practice, but cannot be expected to force such improvements by limiting rights on the basis of a better practice than may be considered reasonable at present. It is desirable to have provisions in decrees for later adjustments in the quantities decreed whenever conditions may warrant such re-determinations. Such provisions will probably only be used at infrequent intervals so as to avoid general disturbance and expense in maintaining water rights.

S. T. HARDING, *Chairman,*

HARRY BARNES,

LYNN CRANDALL,

AUGUSTUS GRIFFIN,

O. W. ISRAELSEN,

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INTERSTATE WATER PROBLEMS AND THEIR SOLUTION*

BY M. C. HINDERLIDER† AND R. I. MEEKER,‡ MEMBERS, AM. SOC. C. E.

SYNOPSIS

The growth and future prosperity of the arid States rest primarily upon the efficient use of the waters of the Western rivers which are a common resource, interstate in character. The use and re-use of these waters from their sources down, is imperative to self-preservation of the States and the welfare of the Nation. Water consumption in the Western United States has attained that stage of development wherein such uses in one or more States frequently give rise to fear of encroachment on the use in adjoining States. Under such conditions wherein the sovereignty of two or more States or nations comes in conflict, it is readily seen that there may be fruitful ground for trouble. The problem is one between States equal in powers and with equal rights of self-preservation, and not one between mere private users whose rights are derived from their respective States.

The rapid development of rivers for municipal, irrigation, and hydro-electric power purposes has brought many interesting engineering and legal problems to the fore during the past fifteen years. None has been more intricate or difficult of solution and settlement than that pertaining to interstate water rights.

Friction between States and nations over the use of interstate or international rivers may easily arise from the fact that political divisions generally do not conform to river-basin boundaries, but cut across the latter in every conceivable direction and overlap on adjacent river basins. Due to physical conditions present in practically all major drainage basins, demands are frequently made on the water supply of one State for use in another State or in another river basin, and hence all phases of water utilization are affected by such conditions. Not the least of the difficulties arising over the administration of interstate streams is that due to differences in State water laws, no two of which are the same. Each State may use the waters of its streams as it may see fit save only for extra territorial burdens imposed, first, by internal treaties; second, by National control of navigation; third, by interstate treaties or compacts; and fourth, by decisions of the United States Supreme Court.

This paper will be confined to interstate river problems of the Western United States, although reference will be made to like problems of the Eastern United States.

NOTE.—Written discussion on this paper will be closed in August, 1926. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Presented at the meeting of the Irrigation Division, Salt Lake City, Utah, July 9, 1925.

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GENERAL STATEMENT

In the arid and semi-arid States in the Western United States, where irrigation is essential to successful agriculture, interstate river problems are becoming more and more apparent, due in part to politics and in part to rapid growth and consequent uses of water.

Table 1 gives a summary of the more important rivers on which irrigation is paramount, and shows the interstate character of these streams. A brief study of the table discloses in a measure the far-reaching effect which questions relating to stream control may have where the interests of sovereign nations are involved. Many tributaries of the rivers noted in Table 1 and other independent streams are also interstate in character.

TABLE 1.—INTERSTATE CHARACTER OF RIVERS IN THE WESTERN UNITED STATES.

River.	States and Nations affected.
Arkansas.....	Colorado and Kansas.
Colorado.....	Wyoming, Utah, Colorado, New Mexico, Arizona, Nevada, California and Mexico.
Columbia.....	Wyoming, Montana, Idaho, Washington, Oregon, and Canada.
Missouri.....	Montana, Wyoming, North Dakota, South Dakota, and Canada.
North Platte.....	Colorado, Wyoming, and Nebraska.
Rio Grande.....	Colorado, New Mexico, Texas, and Mexico.

Until the passage of the Reclamation Act in 1902 irrigation development was chiefly intrastate in character. Then came the era of development under Corporate, Carey, Irrigation District, and Federal Reclamation Acts, each imposing, more or less silently, its demands on Western rivers. As the colonization and development of irrigation projects is a gradual process, there occurs a lagging effect in the utilization of water appropriated for such reclamation, the cumulative effect of which results in reduced water supply which is sometimes not apparent for from ten to fifteen years. It generally happens in Western streams that this condition is only temporary and is largely corrected through return flow to the stream.

Interstate river problems were foreshadowed in the dry cycle of 1902 which, in some river basins, brought this subject of water shortage prematurely to the attention of irrigators, engineers, attorneys, and legislators. A preponderance of wetter years immediately following, together with reservoir development and the healing effect of return flow from irrigated areas, deferred general action until recently. Since the dry cycle of 1902, recurring dry years, in some instances, reflected water shortage in some of the smaller river basins, and pointed the finger of caution to potential interstate conflicts, emphasizing the need of early interstate action. Although in the main, careful studies have proved or will demonstrate the supposed injuries to be chimerical, yet these are causes which will lead to conflict and must be adjusted sooner or later.

Reference to the U. S. Census reports shows the rapid expansion of irrigation during the past twenty years. The figures in Table 2 have been extracted therefrom and are offered for their comparative value.

TABLE 2.—COMPARATIVE DATA. IRRIGATION IN THE UNITED STATES.*

Year.	Acres irrigated.	Capital invested.	Reservoir capacity, in acre-feet.
1889	3 631 000	\$29 534 000	890 000
1899	7 518 000	67 000 000	2 561 000
1902	8 874 000	82 532 000	4 525 000
1909	13 738 000	308 000 000	12 580 000
1919	19 192 000	697 657 000	21 246 000

* See Tables 7, 8, and 15, U. S. Census Report on Irrigation, 1920.

From these data it is quite apparent that reclamation of arid lands reached a culmination in the 1909-19 decade. The effect of such development, together with that of the past five years, on water consumption and return flow is now commencing to be reflected in river flow, especially with the recurrence of dry years, which emphasizes interstate water problems.

Table 3 indicates the growth by Western river basins of irrigated areas for the seventeen-year period of 1902 to 1919, and shows that such areas were increased from 41 to 211 per cent.

TABLE 3.—COMPARATIVE DATA. IRRIGATION IN THE UNITED STATES.

(From U. S. Census Reports.)

River basin.	ACREAGE IRRIGATED.		Percentage of increase, 1902 to 1919.
	1902	1919	
Great Basin streams.....	1 640 000	3 313 000	41
Missouri River and tributaries.....	2 538 000	4 147 000	64
Arkansas River and tributaries.....	393 000	851 000	116
San Joaquin River and tributaries.....	983 000	2 104 000	126
Colorado River and tributaries.....	927 000	2 312 000	149
Rio Grande River and tributaries.....	497 000	1 294 000	161
Columbia River and tributaries.....	1 297 000	3 873 000	198
Sacramento River and tributaries.....	209 000	641 000	211

Reference to similar data on the tributaries of the rivers mentioned in Table 3 shows that in many instances the increase for the seventeen-year period mentioned was frequently as great as 400 and 500%, and even more for areas of considerable size.

Reference to Table 2 shows that the total reservoir capacity in arid and semi-arid States trebled between 1902 and 1909, and more than doubled during the period of 1909 to 1919. Reservoir data by river basins for 1902 are not available for comparative purposes. Table 4 is offered to show roughly the distribution of reservoirs by river basins.

In recent years, prospective and potential irrigation, power, and municipal projects of Western river basins have been outlined more or less by engineering surveys and studies, and from these studies it is quite apparent that future water development will have to be in large units, will be chiefly interstate in character, and will encounter legal and financial difficulties unless interstate

agreements are perfected whereby State titles to river flow are settled. No arid State is immune because State lines do not conform to river basins. Interstate problems concerning municipal water supplies, sanitation, flood control, power development, and related matters are not confined to the West alone; they also occur on rivers of the Eastern United States, thus emphasizing the importance of this subject.

TABLE 4.—DEVELOPED RESERVOIR CAPACITY, WESTERN UNITED STATES: RIVER BASINS FOR YEAR 1919.*

River basin.	Developed reservoir capacity, 1919, in acre-feet.
San Joaquin River and tributaries.....	330 000
Sacramento River and tributaries.....	348 000
Arkansas River and tributaries.....	792 000
Colorado River and tributaries.....	1 676 000
Independent rivers.....	1 899 000
Great Basin drainage.....	2 395 000
Rio Grande and tributaries.....	3 233 000
Missouri River and tributaries.....	4 861 000
Columbia River and tributaries.....	5 712 000
Total.....	21 246 000

* From Table 17, U. S. Census Report on Irrigation, 1920.

Tremendous property values depending on the utilization of interstate streams are at stake, and future water development should be freed from the clouds of possible conflict which act as a bar to the successful financing of such enterprises. Prior to the development of the Compact Method, the only means available for the settlement of interstate controversies was through an appeal to the Supreme Court of the United States.

The experience of Colorado in interstate water litigation has not been an enviable one, although she has never been the aggressor in any such suit. During the period from 1903 to 1925, inclusive, State appropriations for the investigation and protection of Colorado interests in interstate rivers amounted to about \$400 000, and reservoir and ditch companies spent a like amount in joint litigation over such matters. Roughly, a total of \$1 000 000 has been expended by Colorado alone in necessary defense. The interstate problem of the Arkansas River has been in litigation for more than twenty years with still no final settlement, despite one decision by the U. S. Supreme Court, and three suits in the U. S. District Court, two of which are still active. Eleven years elapsed between the date of the inception of, and decision in, the Laramie River case. In addition to the expenditures mentioned, irrigation development in Colorado has been retarded in certain areas or indefinitely postponed, because of interstate litigation. The decisions of the Courts have been unsatisfactory to both sides of each litigated case.

Colorado has been a pioneer on interstate river problems because of her physical location astride the Continental Divide with four major river systems radiating to the four points of the compass. As previously stated, interstate

water litigation has not furnished a satisfactory solution of Colorado's interstate river problems. The compact method for settlement of such problems is the outgrowth of a long period of years of interstate water conflicts.

To the present time interstate litigation over Colorado streams has concerned comparatively small water supplies which are relatively over-developed and yet four States were involved, the streams in question being:

The Laramie River, with an average annual flow at

the Colorado-Wyoming State line of..... 200 000 acre-ft.

The Republican River, with an average annual flow

at the Colorado-Nebraska State line of..... 5 000 " "

The Arkansas River, with an average annual flow at

the Colorado-Kansas State line of about..... 200 000 " "

The heavy expense and long delays incident to securing decisions in these litigated cases, coupled with the general dissatisfaction with the decisions rendered, have pointed to the need of constructive treatment of interstate water problems on the larger rivers where the yearly flow approximates millions of acre-feet, where present and prospective land and other property values run into hundreds of millions of dollars, and where from two to seven States claim joint rights to the natural resource.

In many of the Western river basins where the use of water has not attained a mature stage, comprehensive plans for the greatest use can be made, and conditions are still elastic for interstate adjustments. In some of the smaller interstate river basins the consumption of water has approximated total utilization, and interstate adjustments, whether by litigation or mutual agreements, are increasingly difficult.

ORIGIN OF COMPACT IDEA

The compact method of treatment of interstate river rights was the direct outgrowth of the Laramie River interstate water suit between Wyoming and Colorado. The first compact proposal was made in conjunction with engineering and legal studies of the South Platte River problem in 1916-17; later, in connection with the La Plata River investigations during 1919 and 1920, followed by the Colorado River problems which appeared on the horizon of interstate strife in 1920.

It should be recalled that as far back as 1903 both Elwood Mead, M. Am. Soc. C. E., and Mr. R. P. Teale, of the U. S. Department of Agriculture, directed attention to interstate river problems and discussed the ultimate need of agreements between the States concerning the utilization of interstate waters. Recently, both Herbert Hoover, Hon. M. Am. Soc. C. E., and Mr. Mead have placed the stamp of approval on the interstate water compact principle as a desirable means for the settlement of interstate river problems. It is to Delph E. Carpenter, Interstate River Commissioner of Colorado, however, that the principal credit is due for the working out of the compact idea and the application of it to the settlement of controversies over Western interstate streams.

OBJECT OF RIVER COMPACTS

The purpose of interstate water compacts is to settle the title to river flow between or among the States claiming a river as a common resource. In making compact adjustments, the underlying principles have been to ascertain and define the relative needs and rights as between the States in interest and to safeguard future development against unnecessary delays and the unsettled status of title or wasteful and protracted litigation.

LEGAL BASIS FOR INTERSTATE RIVER COMPACTS: TREATY POWER OF STATES

Each State of the United States is a sovereign power and may legislate as it pleases concerning its internal problems, except as such powers were relinquished under the Constitution. With reference to external problems, Article I, Section 10, Paragraph 3, of the Constitution of the United States recognizes compacts or agreements between States, as follows: "No State shall, without consent of Congress * * * enter into any agreement or compact with another State * * *."

In fact, heretofore, interstate controversies and differences respecting boundaries, fisheries, and other matters, have been frequently settled by interstate compacts through the use of the reserved treaty powers of the States. Among the many boundary disputes thus settled may be mentioned those of Virginia and Pennsylvania, 1780 (11 Pet., 20); Virginia and Pennsylvania, 1784 (3 Dall., 425); Kentucky and Tennessee, 1820 (11 Pet., 207); Virginia and Tennessee, 1802, 1856, (148 U. S., 503, 511, 516); and Virginia and Maryland, 1785 (153 U. S., 155, 162). Compact adjustments of fishery disputes on the Columbia River have been made between Washington and Oregon, and also between Maryland and Virginia in like matters in respect to the Potomac River (153 U. S., 155). For a full discussion respecting the rights of the States to enter into treaties or compacts, with the consent of Congress, see *Rhode Island vs. Massachusetts* (12 Pet., 657, 725-731); *Virginia vs. Tennessee*, (148 U. S., 503); and *Wharton vs. Wise* (153 U. S., 155).

Summarizing, it is found that the States have the same power (with the consent of Congress) to enter into compacts with each other on all matters not delegated to the Federal Government, as independent nations have to make joint treaties.

INTERNATIONAL RIVER PROBLEMS

A review of international river controversies discloses that such matters are usually settled by treaty (Heffter Droit Ind. Appendix VIII; Hall International Law, Sec. 39, 21 Opinion, U. S. Atty. General, 274, 282). The Rio Grande controversy between Mexico and the United States concerning river depletion and water utilization was settled by treaty in 1906. There is also the Canadian treaty of 1909 concerning the present and future use of the waters of the Milk and St. Marys Rivers, the former being a large tributary of the Missouri River. About 1915 an agreement was made in Australia over the waters of the Murray River. The signatories to that agreement were the

States of New South Wales, Victoria, South Australia, and the Commonwealth Government.

An effort has been under way for several years to settle the controversy between Egypt and The Sudan over the waters of the Nile. Likewise, discussions looking to the adjustment of the problems arising over the use of the waters of the Great Lakes and the St. Lawrence River are in active progress between Canada and the United States; and between Mexico and the United States over the Rio Grande below Fort Quitman, Texas.

INTERSTATE RIVER DECISIONS: U. S. SUPREME COURT

The two important Supreme Court decisions concerning the use of Western interstate rivers are those of *Kansas versus Colorado* (May 13, 1907); and *Wyoming versus Colorado* (June 5, 1922).

Kansas versus Colorado.—This interstate river suit arising over the use of the waters of the Arkansas River, was filed by the State of Kansas in May, 1901. The Kansas suit was predicated on the Doctrine of Riparian Rights. In Colorado, the Doctrine of Prior Appropriation prevails. The State of Colorado contended that constitutional provisions entitled it to ownership of all waters within its borders.

The United States intervened in this suit in the interest of its National reclamation policy, and although contending for the Doctrine of Prior Appropriations, raised the question of Federal *versus* State control of water supplies. In substance, the outstanding points of the Supreme Court decision were:

- 1.—It affirmed State ownership and control of waters of non-navigable streams. This was the primary question involved.
- 2.—It defined interstate rights to be an equitable division or apportionment of the benefits arising through the use of waters in an interstate stream.
- 3.—It dismissed the Kansas petition without prejudice on the grounds of insufficient proof of injury.

At the time of the Kansas-Colorado suit (1902), the irrigated area from the Arkansas River and tributaries, according to U. S. Census reports, approximated: In Colorado, 300 000 acres; and in Kansas, 22 000 acres. This interstate river suit occurred during a dry cycle of stream flow and prior to any material development of reservoirs in the Arkansas Basin.

Wyoming versus Colorado.—The Laramie River interstate suit was filed by the State of Wyoming in 1911 over a threatened depletion of the Laramie River by a trans-mountain canal system and tunnel. Many conjectures have been made and several fallacies have obtained concerning the Doctrine of Priority, and its application to interstate river problems. Perhaps the chief fallacy concerning this doctrine and its application to interstate streams has been the assumption that administration of river flow should be basin-wide in character regardless of State lines, and that such administration should be by the Federal Government.

The far-famed Laramie River Supreme Court decision of June, 1922, has been repeatedly cited as sustaining this contention. The Laramie River

decision was based on the fundamental principles underlying the Doctrine of Priority, but the respective private rights in each State were grouped and the allocation made was based on the grouped demands and not on a basin-wide administration of priorities of both States. The Court gave to the most junior project (Colorado Tunnel) a preferred right of diversion as against senior canals in Wyoming, which were compelled to build reservoirs to supply the deficiency. No Federal bailiff was appointed and no super-administration placed in the hands of the Federal Government. The internal administration of priorities of the Laramie River in Colorado or in Wyoming remains undisturbed. No interposition of Colorado priorities was plastered upon the Wyoming priorities, or *vice versa*. In substance, 43 000 acre-ft. per year, of the Laramie River water supply, was allocated to Colorado, wet and dry years alike, if obtainable. The remainder of the river flow was allocated to Wyoming, and the burden of reservoir regulation placed on Wyoming users.

In substance, the Wyoming-Colorado interstate river decision was as follows:

- 1.—It affirmed the fundamental principle of priority as the proper basis for allocating river flow to established uses in each State when both States recognize the same doctrine, but re-affirmed the Kansas-Colorado decision where local laws differ.
- 2.—It held untenable the Colorado contention that a State may use all the water of an interstate stream which originates within its boundaries.
- 3.—It held untenable the Wyoming contention that appropriations of Laramie River water should be limited to use in the Laramie River watershed.
- 4.—It declined to pass on the Government contention that all unappropriated water of Western streams belonged to the United States and are wholly removed from State control.
- 5.—It held that the burden of reservoir development is on the lower State.

The Doctrine of Priority is an intrastate doctrine, is primarily a rule of local administration, and its application in a sweeping way to a large river basin affecting several States where various water rights date back as far as fifty years, would disturb water rights in the several States and attendant large property values. Furthermore, the transition from semi-arid to humid conditions in a lower State whereby a different rule would apply, would make basin-wide administration impracticable.

Physical conditions and the element of time in river flow are also opposed to operating vast river basins (such as those of the Colorado or North Platte River), hundreds of miles in length, with hundreds or, perhaps, thousands of ditches diverting water, according to priority regardless of State lines. Such vast river basins would necessarily have to be sectionized and water administration localized, which State lines do automatically. Another objection to the administration of river basins as a whole, according to the Doctrine of Priority, is that changing conditions and the pressure of need may change the present method and character of water administration within a State.

Any means of interstate settlement should avoid disturbance of the internal relation of State priorities.

The application of the Doctrine of Priority as a principle in determining interstate apportionment is, however, sound. Such an apportionment was made in the Laramie River case between the States of Wyoming and Colorado where State priorities were lumped in determining each State's quota. Definite quantities per annum were allocated to the upper State, the remainder going to Wyoming, the lower State. State administration was left intact. No Federal administration was imposed on the Laramie River, and none is necessary on that or any other interstate river. There has been a growing tendency to demand too much Federal assistance on interstate relations to the detriment of State autonomy. The theory of Federal administration of interstate rivers is an apt illustration of such tendencies. As recently as May, 1925, President Coolidge pointed out the fallacy of too much Federal intervention in State affairs and the need of more State responsibility in interstate matters.

With reference to State or Federal control of interstate streams, in 1903,* Mr. Mead commented as follows:

"It would be unscientific and in the highest degree unfortunate if the principle of local self-government, on which this nation is founded, and the opportunity to exercise self-reliance and self-control, which has done so much for its manhood in the past, should be taken away from the irrigators of the West by the transfer of the local regulation of streams to some centralized bureau."

Under the compact method of settlement, State autonomy in water matters is preserved and intrastate priorities undisturbed as to position. In compact adjustments of river supplies where development and water utilization are far advanced, definite State allotments are necessary. In river basins where water utilization is in its infancy, State allocations can be made more or less elastic to fit changing conditions and the needs of the future.

Under the litigation method of settlement of interstate water disputes, all efforts are centered on winning the suit. A vast amount of information is submitted in an unco-ordinated way, and inadequate attention given to potential water needs.

Under the compact or treaty method of adjustment, opportunity is had to study a river basin as a whole, and from such study to arrive at some amicable settlement based on established facts as to total water supply, irrigated and irrigable areas, municipal necessities, reservoir opportunities, power possibilities, and future development.

Such studies must necessarily include questions of the relative necessities of each State as determined by climatic conditions, environment, natural resources, and facilities for utilizing them in the most efficient manner, ever bearing in mind that for all time to come Nature has placed a limit on a common resource far below the requirements, thereby making it incumbent on all those who would avail themselves of its use to conserve and re-use this resource in the most efficient way. A lower State which permits water, the very life-blood of an upper State, to flow unused through the lower State should not be heard

* "Irrigation Institutions," p. 378.

to complain of any unfair apportionment of the benefits arising out of the use of that stream by the upper State. To hold otherwise would be abhorrent to all the theories of justice, equity, and economics, as well as the fundamental principles on which American irrigation institutions have been founded. Owing to climatic differences and accidents of settlement in a river basin development has frequently occurred first in the lower States. Development on the head-water States is more gradual, and if interstate rights are determined entirely by priority rule, the upper State might be largely deprived of a natural resource having its origin principally in its own territory. It would be impossible to remedy this evil by the exercise of eminent domain because such exercise would be limited by the boundaries of the upper State and could not affect property in the lower States. The upper State is, therefore, left one of two alternatives: First, to prevent development in the lower State; or, second, to secure a recognition of its rights for future development by interstate agreement or compact.

RECENT INTERSTATE LEGISLATION

Recent legislation by the States of Wyoming and Utah provides for cooperation with adjacent States in the matter of acquiring, determining, supervising, and regulating water and water rights of interstate streams. This is a constructive step destined to have a considerable field of application in the future. Oregon has older similar reciprocal legislation concerning the State line canals, but it is less broad than other features of the Utah Act of 1921.

Within the past five years, several States and Federal agencies have entered into joint engineering studies of all phases of water utilization of several intrastate and interstate rivers. Comprehensive reports with recommendations have been made, and those on interstate streams will probably be the basis of future water agreements between the States affected and the Federal Government.

Reference is made to the study and report on the Columbia River Basin published by the Federal Power Commission in 1923. The States of Washington, Idaho, and Montana, and several departments of the Federal Government participated in this study.

During the past winter (1924-25), North Dakota has passed legislative acts providing for two interstate water commissions—the Missouri River Commission to be composed of representatives of Montana, South Dakota, and North Dakota, and the Red River Flood Commission which provides for participation by Minnesota, South Dakota, and North Dakota.

HISTORICAL REVIEW OF INTERSTATE RIVER COMPACTS

The object of interstate water compacts is to determine the respective rights of the various interested States to the use of a common river supply. The compact idea of interstate river apportionment was formulated in 1916 and 1917 in connection with legal and engineering studies of the South Platte River in a controversy then pending between the States of Nebraska and Colorado. The first application of the compact plan of settlement, however,

occurred on two other interstate rivers, namely, the Colorado River and the La Plata River. The necessary State and National legislation providing for interstate river compacts was passed in 1921, and the first compacts were drafted in November, 1922, for the Colorado River and the La Plata River, at Santa Fé, N. Mex.

A marked contrast occurs on these two interstate river problems. The Colorado River water supply involves seven States, 20 000 000 acre-ft. of water per year, water rights to 3 000 000 acres of irrigated land, and large potential irrigation and power possibilities. Water utilization is still far from having attained a complete stage of development in the Colorado River Basin, and about 13 000 000 acre-ft. per year passes unused to the Pacific.

The La Plata River involves two States, 23 000 acres of irrigated land, and about 60 000 acre-ft. of water per year, with the direct-flow rights heavily over-decreed, and with storage as the final step in water utilization.

Six years after the commencement of engineering studies on the La Plata River, and four years after the initial legislative act, an interstate compact is in operation. The Colorado River Compact is still an open problem, although five of the seven States have ratified unconditionally and one conditionally. Considerable progress has been made when it is realized that engineering studies usually require two or more years, legislative ratification can only occur at two-year intervals, and Congressional ratification can seldom be accomplished in less than one year.

In a comparative way the Colorado River problem may be classed as a major one for accomplishment, and the La Plata River a minor interstate problem when volume of water is considered, although the La Plata River Problem required a large amount of study and effort to reach a settlement. The South Platte Compact is now awaiting Congressional ratification.

Interstate compact accomplishments may be briefly summarized as follows (status as of May, 1925):

River compacts concluded, under negotiation, or provided for by legislative act.....	10
River compacts concluded.....	5*
River compacts ratified by interested States, with or without reservations.....	5
River compacts in operation.....	1

Table 5 gives the salient features of rivers and States affected by compacts concluded, under negotiation, or provided for by legislative act.

PROSPECTIVE INTERSTATE WATER PROBLEMS

A review of water utilization and prospective development on other interstate rivers discloses many nascent and potential interstate water problems. Among these may be mentioned the following (Table 6), although there are undoubtedly many others.

* Includes the Delaware River Compact between New York, New Jersey, and Pennsylvania.

TABLE 5.—STATUS OF INTERSTATE RIVER COMPACTS, MAY, 1925.

River.	States affected.	Status.
WESTERN UNITED STATES		
Arkansas.....	Colorado and Kansas.....	Negotiations in progress.
Colorado.....	Arizona, California, Colorado, Nevada, New Mexico, Utah, and Wyoming.....	Compact concluded. Ratified intact by all States except Arizona and California, which attached reservations.
La Plata.....	Colorado and New Mexico.....	Compact concluded. Ratified by both States; ratified by Congress; signed by President, January 29, 1925. In operation season of 1925.
North Platte.....	Colorado, Nebraska, and Wyoming.....	Engineering investigations completed; hearings held in 1924 in three States; active negotiations under way.
Pecos.....	New Mexico and Texas.....	Compact concluded December 19, 1924; amended February 10, 1925.
Rio Grande.....	Colorado, New Mexico, and Texas.....	Engineering investigations completed; hearings scheduled for 1925.
South Platte.....	Colorado and Nebraska.....	Compact concluded; ratified by both States; awaits Congressional ratification.
EASTERN UNITED STATES		
Delaware.....	New York, New Jersey, and Pennsylvania.....	Compact concluded. New Jersey: Referred to a Board of Seven for consideration to report to 1926 Legislature. Pennsylvania: Passed Senate; amended in Assembly with reservations proposed by New Jersey. NOTE.—Ratification by Pennsylvania impossible before 1927. Compact as amended passed Assembly, but failed in Senate. New York: Compact with reservations and amendments proposed by New Jersey passed without opposition, and signed by Governor Smith on March 18, 1925.

TABLE 6.—INTERSTATE WATER PROBLEMS.

Stream.	States affected.	Nature of problem.
SURFACE STREAMS.		
James River.....	North and South Dakota....	Flood control and drainage.
Big Sioux River.....	South Dakota and Iowa.....	Flood control and drainage.
Columbia River.....	Montana, Idaho, Washington, and Oregon.....	Irrigation and power chiefly. Columbia Basin Project involved.
Snake River.....	Colorado and Wyoming.....	Irrigation uses.
South Canadian River.....	Arkansas, Kansas, New Mexico, and Oklahoma....	Flood control and conservation.
Arkansas River.....	Arkansas, Kansas, and Oklahoma.....	Flood control and conservation.
Grand River.....	Kansas, Missouri, and Oklahoma.....	Flood control and conservation.
Caney River.....	Arkansas, Kansas, and Oklahoma.....	Flood control and conservation.
UNDERGROUND WATER.		
Artesian Basin.....	North and South Dakota....	Regulation of present waste from about 10 000 artesian wells.

RÉSUMÉ

A résumé of the foregoing discussion, it is believed, will reveal the following pertinent and outstanding facts:

First.—The vast importance of the subject herein discussed as affecting relationship between sovereign States, the protection of existing rights in each State, and the systematic, necessary, and logical development of the latent resources of not only the Western United States, but all parts of the Nation where such resources are of an interstate character.

Second.—That with respect to the status of interstate streams, the Supreme Court of the United States has made a number of well-defined rulings which have quite definitely established certain limitations to the exercise of State and Federal control of a resource common to two or more States, and which point the way for future action in interstate matters. These may be enumerated as follows:

- (a) State ownership and control of waters of non-navigable streams.
- (b) Equitable apportionment between States of the benefits arising through the use of the waters of an interstate stream.
- (c) Recognition of the fundamental principle of priority as a proper basis for allocating river flow to users claiming water from an interstate stream when the interested States recognize the common doctrine of priority of use.
- (d) Holding as untenable the theory that a State may rightfully claim the exclusive use of all the waters originating within the boundaries of such State to the serious detriment of a lower State where such water supply is a common resource, and where each State recognizes the doctrine of appropriation.
- (e) Affirms the principle that the locus of application of the waters of a stream common to two or more States is not limited to the drainage basin of such stream.
- (f) Among other things the Supreme Court has refused to pass upon the contention of the U. S. Government that all the unappropriated waters of the Western streams belong to the United States and are wholly removed from State control, but a previous ruling does recognize the States' control of such waters.

As a result of such decisions it is apparent that no State is supreme in matters pertaining to the utilization of the waters of a stream common to two or more States; that there must be recognition of a common right of all States concerned to share therein. It naturally follows that some means must exist or be found for the determination and allocation of the benefits arising under such right. As between two nations, such means are limited to arbitration or war. As between the States forming the nation, such means are limited to arbitration or to appeals to the U. S. Supreme Court, resort to force being prohibited under the Constitution. The treaty or compact method would seem to offer great possibilities and a rational basis for the settlement of interstate disputes, as such method provides ample opportunity for a dispassionate analysis of all conflicting claims and equities, in an atmosphere of fairness and in the light of all established facts.

It is the opinion of the writers that the opportunities for the adjustment of such disputes, offered by such means, should be resorted to first, and that only after every effort has been completely exhausted should either State appeal to the Court of last resort. It is believed that such procedure is scientifically and economically sound and is certainly conducive to that sense of interstate comity which should prevail between sister States as one of the requisites of Government stability.

Any decision based on an incomplete presentation of the facts, or a perversion of the same, which is likely to occur in any Court proceeding, and which leaves the losing party honestly convinced of the unfairness of the decision, is fundamentally wrong, and in a true sense, cannot be regarded as a real settlement of the dispute. Court decisions follow Constitutional provisions and statutes. The former of necessity are fundamental in their application and may permit of considerable variation in interpretation. The latter, although supposedly based on the elements of equity, justice, and common sense, often fall far short of the objective sought, and the Court interpretation of such is frequently susceptible of many shadings.

As long as conditions which permit of litigation between States are allowed to exist through failure to provide a more rational method for the elimination of such disputes, expensive litigation resulting in an unsettled state of affairs will continue as a clog on future development. Suits between individuals or corporations of different States are expensive, both in time and money, and even when decisive, the results are limited to the parties in action and to the particular question before the Court. Suits between States which must be brought in the U. S. Supreme Court usually result in more or less definite determinations, but such decisions require long periods of time and the expenditure of large sums of money. Some of the decisions indicate that the Court was more interested in enunciating some principle applicable to the case in question than in a definite and constructive settlement of the differences at issue.

On the other hand, under the treaty method of adjusting interstate controversies, full play is given for the application of constructive methods looking to a realization of the legitimate aspirations of each party. Furthermore, when a common ground has been found, the consummation of a compact between States eliminates all future litigation of the matters treated, not only between the States, but between the residents thereof as well, thereby promoting orderly development through the determination of the relative rights to the use of a common resource under conditions to be determined by the owners thereof.

Furthermore, a treaty between sovereign powers is not susceptible of arbitrary change or revocation or modification without the consent of the contracting parties, which tends for greater stability as against Court decisions which are susceptible of reversal or such modifications as may seem meet and proper to the Court. On the other hand, in the drafting of an interstate compact, provision may be made by the contracting parties for desirable modification which might appear necessary as time passes, which would provide a

flexible and practical method of administration, compatible with the necessities of an advancing age. Such conditions would not ordinarily obtain under the inflexible rules and limitations which usually hedge about the administration of a Court, decree.

Not the least of the advantages to be secured to each State through the use of the compact method of settling interstate disputes is the elimination of a real menace to State institutions and constitutional guaranties, that of Governmental interference in the allocation and administration of public water supplies. The writers know of no theory advanced by proponents of Federal control of the resources of Western States that would be so inimical to the welfare of irrigation institutions as that of Governmental supervision of the administration or control of public water supplies. If for no other reason than this alone it is highly important that the Western States adopt some method for the adjustment of controversies over interstate streams which would eliminate the necessity for an appeal to a Federal tribunal with the consequent opportunity for imposing Federal supervision.

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In general, the more uniform the flow of a stream the more useful it is in its natural state, whereas the greater its variation the greater the necessity for reservoir regulation. Uniformity of flow is, however, of secondary consideration in water sheds where there are feasible reservoir facilities to conserve the entire supply.

In the arid West there is much more land suited for irrigation than there is water to supply it. Practically all land that can be irrigated from the reservoir supply has been under irrigation for many years. The expansion of irrigated areas, therefore, must depend in the main on reservoir water supply. On this account it is safe to state that in the arid West the important river characteristic is total annual water yield rather than uniformity of flow.

NOTE.—Written discussion on this paper will be closed in August, 1925. When finally closed the paper with discussion in full will be published in Transactions.

* Presented at the meeting of the Power Division, Salt Lake City, Utah, July 9, 1925.

STREAM REGULATION WITH REFERENCE TO IRRIGATION AND POWER*

By J. C. STEVENS,† M. Am. Soc. C. E.

SYNOPSIS

The regulation of the water of a stream to render it available for both power and irrigation is largely a local problem.

Power and irrigation demands are usually so widely different that a decision must be reached as to which use shall be given the preference, and rarely do the two demands dovetail so completely that there is no conflict.

In some sections, notably in California, a source of power is being rapidly developed that is purely a by-product of irrigation. Where large reservoirs are constructed for the storage of irrigation water, plants are being built to develop power when and as irrigation water is released from them. On some streams by reason of the law of priority of water rights the opposite is true and irrigation use is secondary to power, agriculture by irrigation thus becoming a by-product of industry.

The purpose of this paper is to inquire in general terms into the varied conditions where power and irrigation uses are of importance and to ascertain whether it is possible to aid in providing against a conflict of interests where new developments are in prospect, or in harmonizing to some extent conflicts that already exist.

STREAM FLOW CHARACTERISTICS

In general, the more uniform the flow of a stream the more useful it is in its natural state, whereas the greater its variation the greater the necessity for reservoir regulation. Uniformity of flow is, however, of secondary consideration in water-sheds where there are feasible reservoir facilities to conserve the entire supply.

In the arid West there is much more land suited for irrigation than there is water to supply it. Practically all land that can be irrigated from the unreserved supply has been under irrigation for many years. The expansion of irrigated areas, therefore, must depend in the main on reservoir water supply. On this account it is safe to state that in the arid West the important river characteristic is total annual water yield rather than uniformity of flow.

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† Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

It is obvious that if the total annual yield of a water-shed is to be used for irrigation, sufficient storage capacity must be provided to make that supply available during the irrigation season, and it makes no great difference whether that supply accrues gradually throughout the year or within a few months.

IRRIGATION DEMAND

Passing from the southern to the northern limits, or higher altitudes, of the arid West the irrigation season becomes shorter, with correspondingly greater variation in the monthly irrigation demand. In order to illustrate this point the water requirements for typical irrigated sections of the West are shown in Fig. 1. It will be noted that the maximum monthly demand varies from 13% of the yearly total in the Imperial Valley to 34% in the Klamath District. It is obvious, therefore, that for equal land areas and equal yearly water yields, reservoir capacities and distributing canal capacities

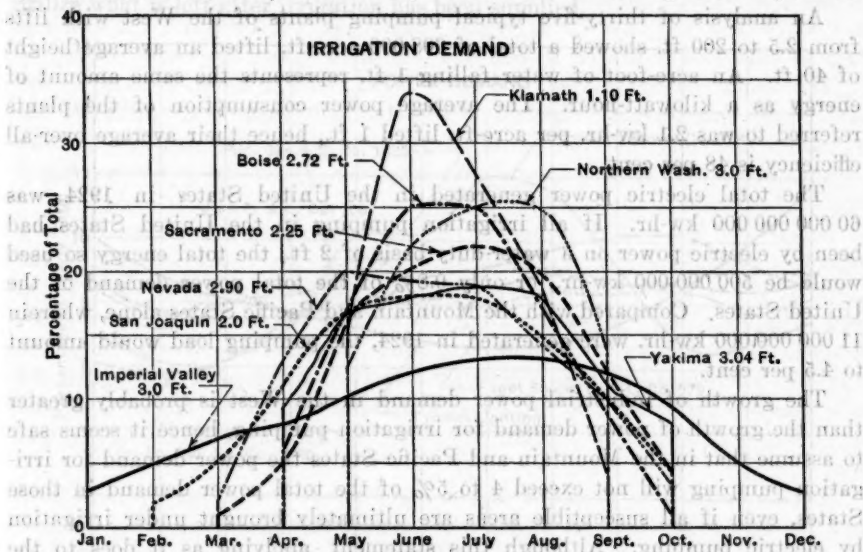


FIG. 1.

must be relatively greater as we go northward, if the same yearly total of irrigation water is applied. There appears to be no relation between the net yearly duty of water and the intensity of its application, this being governed by factors other than climatic. For example, the water requirements of Imperial Valley with a 12-month irrigation season are no greater than those of Northern Washington with only a 5-month season; but the intensity of application in the northern district is double that in the southern, hence for equal water supplies, reservoir and canal capacities per acre in Northern Washington must be twice those of Southern California to give the desired degree of regulation.

POWER DEMAND

The only common characteristic between irrigation and power demands is the irrigation pumping load. Although considerable power in the aggre-

gate is used for irrigation pumping the proportion of total generated power so used in the West is so small that its effect on the total power demand is hardly noticeable. That power for irrigation pumping is destined, however, to become a much more important factor than at present is evident from the continually increasing pumping demand.

There are about 19 000 000 acres under irrigation at present in the United States, of which 16 000 000 acres, or 84%, are supplied by gravity, and 3 000 000 acres, only 16%, by pumping. It has been estimated that there are approximately 15 000 000 additional acres that can be irrigated by gravity through reservoir water supplies, and 10 000 000 additional acres that can be watered by pumping. Thus, the proportion of acres irrigated by pumping may be expected to increase from 16%, as at present, to 38% as the ultimate limit.

An analysis of thirty-five typical pumping plants of the West with lifts from 2.5 to 200 ft. showed a total of 603 000 acre-ft. lifted an average height of 40 ft. An acre-foot of water falling 1 ft. represents the same amount of energy as a kilowatt-hour. The average power consumption of the plants referred to was 2.1 kw-hr. per acre-ft. lifted 1 ft., hence their average over-all efficiency is 48 per cent.

The total electric power generated in the United States in 1924 was 60 000 000 000 kw-hr. If all irrigation pumping in the United States had been by electric power on a water-duty basis of 2 ft., the total energy so used would be 500 000 000 kw-hr., or only 0.8% of the total power demand of the United States. Compared with the Mountain and Pacific States alone, wherein 11 000 000 000 kw-hr. were generated in 1924, the pumping load would amount to 4.5 per cent.

The growth of industrial power demand in the West is probably greater than the growth of power demand for irrigation pumping, hence it seems safe to assume that in the Mountain and Pacific States the power demand for irrigation pumping will not exceed 4 to 5% of the total power demand in those States, even if all susceptible areas are ultimately brought under irrigation by electric pumping. Although this statement, applying as it does to the entire West, indicates that irrigation pumping will not be a controlling factor in power development, it is, nevertheless, a very important factor in certain localities.

Fig. 2 shows the monthly power output of five large power systems of the West. The demand of the Idaho Power Company shows the greatest relative demand for irrigation pumping, with the Southern California Edison Company second. For comparison a typical lighting load is shown on the same diagram. Note that the load characteristics of the Idaho Power Company are quite symmetrically opposite to those of a lighting load, due almost exclusively to irrigation pumping.

RELATION OF WATER SUPPLY TO IRRIGATION AND POWER DEMAND

Practically none of the rivers of the country has flow characteristics that coincide with either the power or the irrigation demand. It is obvious, there-

fore, that if the natural water supply is to be utilized as fully as practicable, extensive artificial regulation is inevitable.

Figs. 3 to 9, inclusive, show the water supply characteristics in seven representative districts, in relation to the irrigation and power demands therein. Quantities are expressed in percentage of yearly totals, hence equal areas are enclosed by each curve.

Irrigation is generally recognized as the superior use of water, and rightfully so. Water supply alone governs the extent of irrigation, but does not govern solely the amount of power that may be developed, as both head and supply are factors in hydro-electric developments. There are many substitutes for water in power generation, but there is no substitute for water in irrigation. It may as well be conceded, therefore, that the water of the West is destined to be used for irrigation primarily and that power generation may utilize what is left after irrigation has been supplied.

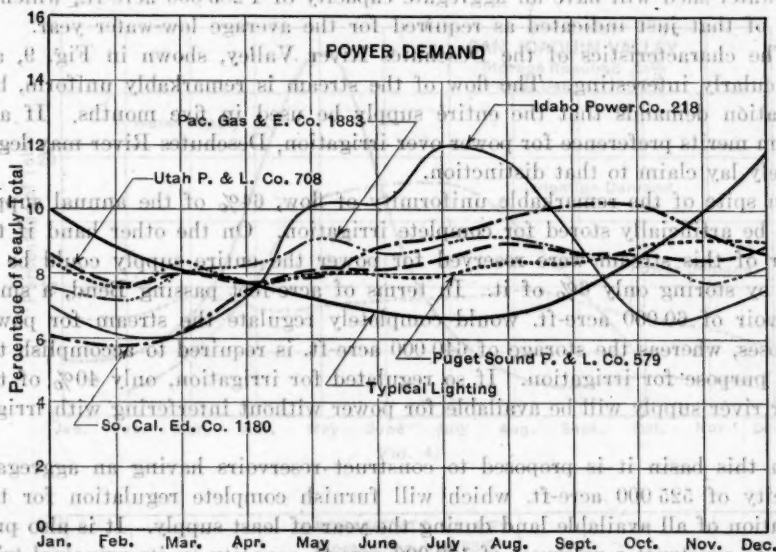


Fig. 2.

On each of the diagrams, Figs. 3 to 9, the extent of artificial storage required for complete utilization of the supply for irrigation is indicated in percentage of the yearly supply, also the proportion of the annual water supply that can be used for power after irrigation requirements have been satisfied.

Fig. 3 shows the characteristics of the Lower Colorado River Valley. Of the flow of the Colorado River, 26% will have to be stored to render the entire supply available for irrigation. This would amount to 2 800 000 acre-ft. for the average low-water year. The U. S. Bureau of Reclamation has placed 2 300 000 acre-ft. as the limit of storage required for all lands susceptible of irrigation in both the Upper and Lower Basins, as there is not quite sufficient land to utilize the entire water supply. If this river were completely regulated by storage for irrigation, 85% of the water supply could be used

for power without interfering with irrigation. This would mean the possible distribution of 700 000 000 kw-hr. for every 100 ft. of fall utilized.

Fig. 4 shows the characteristics in the San Joaquin Valley. The storage required is 34% of the yearly supply. Of this supply 65% could be used for power without interference.

Fig. 5 shows the characteristics of the Sacramento Valley and indicates that 63% of the annual supply will have to be artificially stored for complete utilization of the water supply for irrigation. Only 58% of the supply may be used for power without interference.

In the Salt Lake Basin, Fig. 6 shows that 64% of the water must be stored for irrigation and that only one-half the yield may be used for power.

As shown in Fig. 8, 44% of the supply in the Yakima Valley must be stored for complete irrigation, and power use may be made of 58% of the yearly supply. Reservoirs already constructed or planned for early completion in this water-shed will have an aggregate capacity of 1 285 000 acre-ft., which is 90% of that just indicated as required for the average low-water year.

The characteristics of the Deschutes River Valley, shown in Fig. 9, are particularly interesting. The flow of the stream is remarkably uniform, but irrigation demands that the entire supply be used in five months. If any stream merits preference for power over irrigation, Deschutes River may legitimately lay claim to that distinction.

In spite of the remarkable uniformity of flow, 64% of the annual supply must be artificially stored for complete irrigation. On the other hand if the water of this stream were reserved for power the entire supply could be so used by storing only 6% of it. In terms of acre-feet passing Bend, a small reservoir of 60 000 acre-ft. would completely regulate the stream for power purposes, whereas the storage of 640 000 acre-ft. is required to accomplish the same purpose for irrigation. If so regulated for irrigation, only 40% of the upper river supply will be available for power without interfering with irrigation.

In this basin it is proposed to construct reservoirs having an aggregate capacity of 525 000 acre-ft. which will furnish complete regulation for the irrigation of all available land during the year of least supply. It is also proposed to construct a reservoir of 100 000 acre-ft. capacity on its principal tributary, Crooked River, to be used entirely for power on the lower river, thus regaining to some extent the power possibilities lost by giving Deschutes River water preferential irrigation use.

It should be emphasized that the foregoing data and deductions are in general terms, and are presented to show the maximum characteristic tendencies only, without in any way attempting to outline specific projects or supplanting particular studies that have been made.

In particular the data given concerning the quantity of water available for power after irrigation requirements have been met are subject to a variety of interpretations. If one group of power plants could be placed above irrigation reservoirs and another group below, but above canal diversions, in many cases the entire supply might be almost completely utilized for both power and irrigation without conflict of interest. In some instances, for example

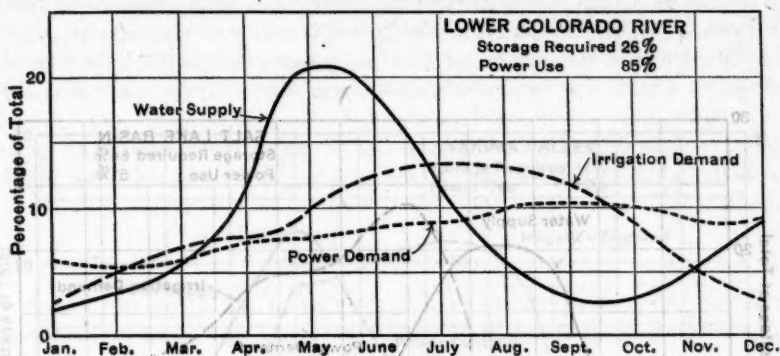


FIG. 3.

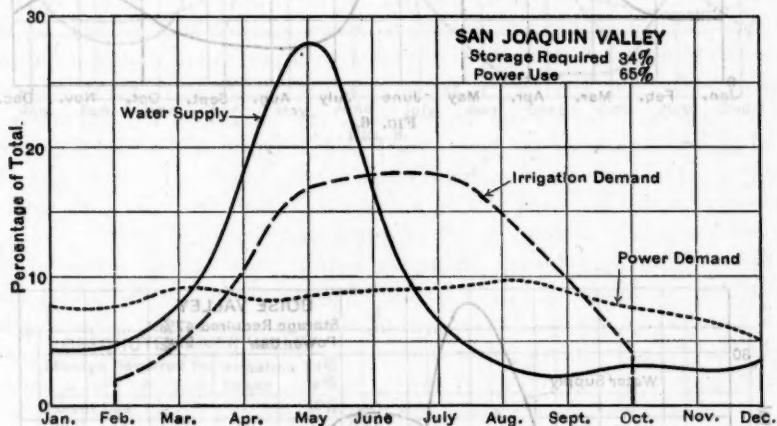


FIG. 4.

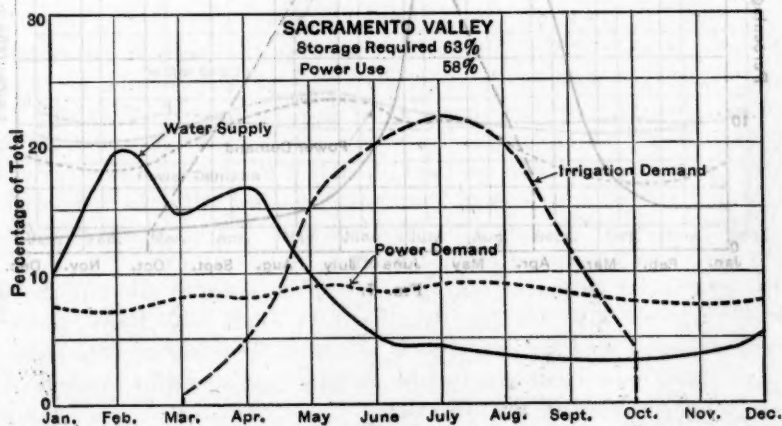
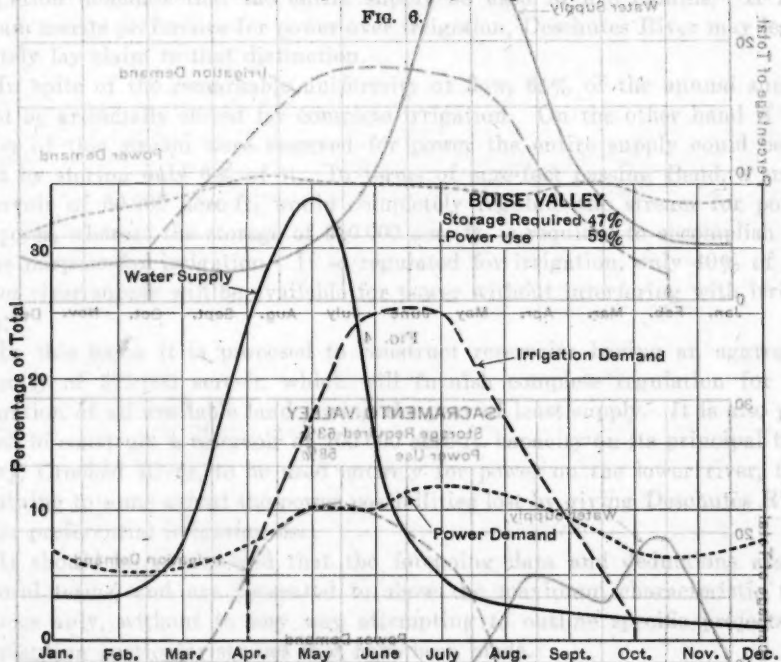
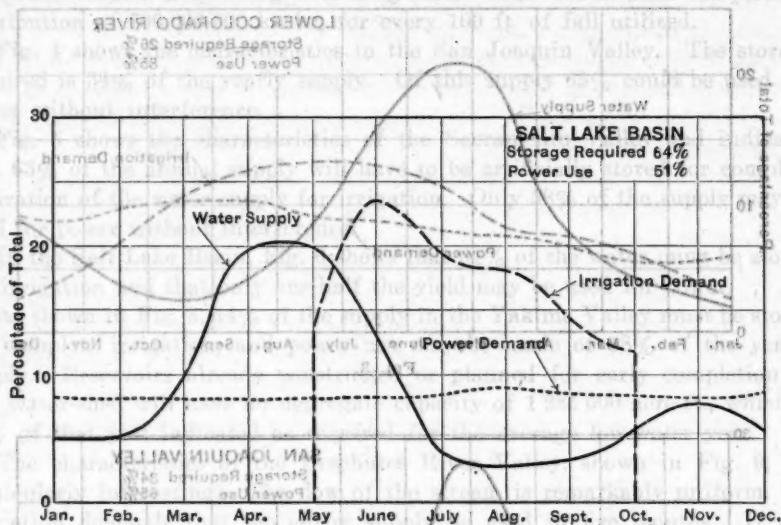


FIG. 5.



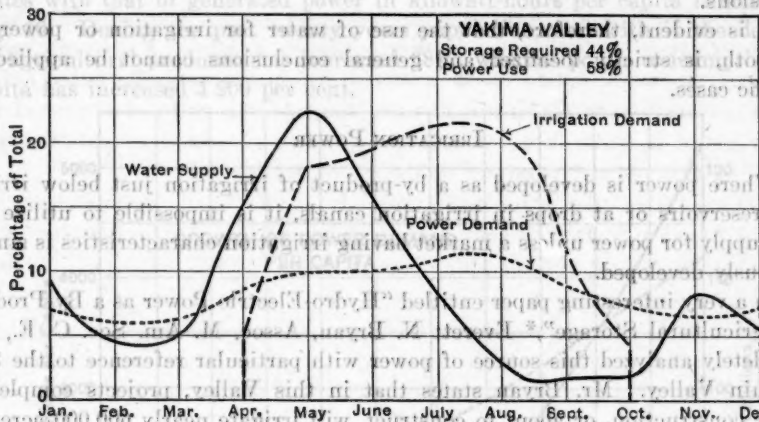


Fig. 8

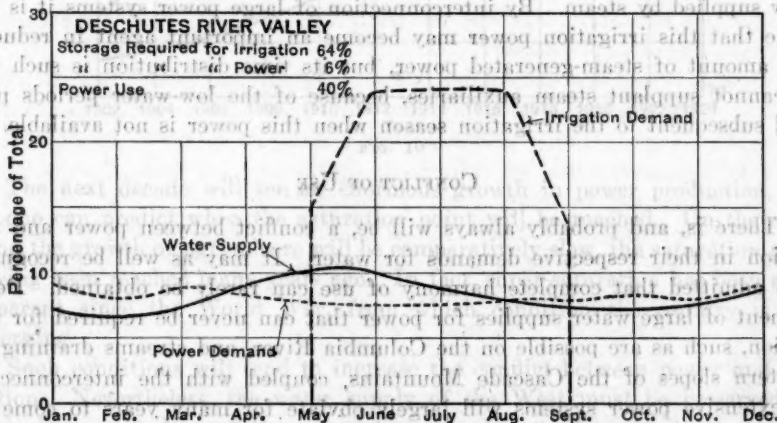


Fig. 8

on Deschutes River, power development will be largely confined to the lower river where return water from irrigation becomes a very important factor. On other streams, notably in the Sierras, power plants with their own storage reservoirs, are being extensively developed above irrigation reservoirs and diversions.

It is evident, therefore, that the use of water for irrigation or power, or for both, is strictly localized and general conclusions cannot be applied to specific cases.

IRRIGATION POWER

Where power is developed as a by-product of irrigation just below irrigation reservoirs or at drops in irrigation canals, it is impossible to utilize the full supply for power unless a market having irrigation characteristics is simultaneously developed.

In a very interesting paper entitled "Hydro-Electric Power as a By-Product of Agricultural Storage",* Everett N. Bryan, Assoc. M. Am. Soc. C. E., has completely analyzed this source of power with particular reference to the San Joaquin Valley. Mr. Bryan states that in this Valley, projects completed, under construction, or about to construct, will irrigate nearly 600 000 acres of land by gravity and at the same time sufficient power will be developed to irrigate an equal area by pumping through a 90-ft. lift. It is obvious that if irrigation water is stored 100 ft. above the Valley, the release of that water represents sufficient energy to lift an equal quantity 70 to 80 ft.

The utilization of such irrigation power must depend quite largely on irrigation pumping, or otherwise on its transmission to industrial markets now supplied by steam. By interconnection of large power systems it is possible that this irrigation power may become an important agent in reducing the amount of steam-generated power, but its time distribution is such that it cannot supplant steam auxiliaries, because of the low-water periods prior and subsequent to the irrigation season when this power is not available.

CONFLICT OF USE

There is, and probably always will be, a conflict between power and irrigation in their respective demands for water. It may as well be recognized and admitted that complete harmony of use can rarely be obtained. Development of large water supplies for power that can never be required for irrigation, such as are possible on the Columbia River, and streams draining the western slopes of the Cascade Mountains, coupled with the interconnection of extensive power systems will largely obviate for many years to come the necessity of an attempt to harmonize these conflicting uses.

In particular instances considerable can be done to dovetail the use of water for power with that for irrigation. The Salt Lake Basin, where practically all water used for irrigation is also used for power at some point in its line of travel from mountain to lake, furnishes an excellent example of what may be accomplished in this direction.

* Transactions, Am. Soc. C. E., Vol. 88 (1925), p. 910.

The demand for power is increasing much more rapidly than the demand for agricultural products. The demand for foodstuffs increases directly with increase of population, whereas demand for power per capita is compounding at a startling rate. A comparison of the growth of population in the United States with that of generated power in kilowatt-hours per capita is shown in Fig. 10. During the past twenty years population, and with it the demand for agricultural products, has increased 38%, whereas power consumption per capita has increased 1 200 per cent.

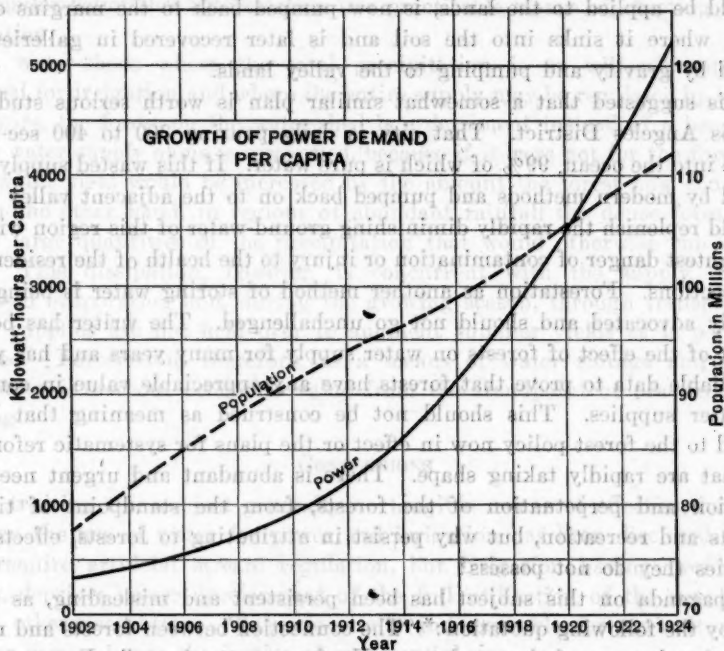


Fig. 10.

The next decade will see an enormous growth in power production, and no one can predict when the saturation point will be reached. On the other hand the growth of agriculture will be comparatively slow, the saturation point having been reached many years ago. In fact, super-saturation has been quite apparent since the World War, from which condition the country is just emerging.

Such conditions will tend to increase the conflict between power and irrigation. Nevertheless, the water supply of the West must be preserved for agriculture regardless of the power demand. Every effort should be made to harmonize the two wherever conditions permit. It is believed that much can be done to accomplish this, but the problem is strictly a local one and no general plan of procedure can be outlined.

METHODS OF REGULATION

Strictly speaking there is only one known method of regulating stream flow, and that is by storage. Storage, however, may be accomplished in more than

one way. The storage of water behind dams in suitable reservoir sites is the most direct and efficient method known. It is estimated that in the West suitable reservoir sites exist where water supply sufficient for the irrigation of 15 000 000 additional acres may be developed.

In some localities where suitable reservoir sites are not to be found extensive soil storage may be developed. In Butte Valley, Northern California, water from melting snows that formerly collected into basins and evaporated before it could be applied to the lands, is now pumped back to the margins of the Valley where it sinks into the soil and is later recovered in galleries and applied by gravity and pumping to the valley lands.

It is suggested that a somewhat similar plan is worth serious study for the Los Angeles District. That city is daily pouring 300 to 400 sec.-ft. of sewage into the ocean, 99% of which is pure water. If this wasted supply were treated by modern methods and pumped back on to the adjacent valley lands it would replenish the rapidly diminishing ground-water of this region without the slightest danger of contamination or injury to the health of the residents of those sections. Forestation as another method of storing water is being persistently advocated and should not go unchallenged. The writer has been a student of the effect of forests on water supply for many years and has yet to find reliable data to prove that forests have any appreciable value in conserving water supplies. This should not be construed as meaning that he is opposed to the forest policy now in effect or the plans for systematic reforestation that are rapidly taking shape. There is abundant and urgent need for protection and perpetuation of the forests, from the standpoint of timber products and recreation, but why persist in attributing to forests, effects and properties they do not possess?

Propaganda on this subject has been persistent and misleading, as illustrated by the following quotation: * "The connection between forests and rivers is like that between father and son. No forests, no rivers." Exaggeration, of course, but nevertheless an epitome of the intellectual diet on which engineers have been fed.

Great plans have been made for exhaustive research into this question. Comparable areas, forested and treeless, but otherwise similar, have been selected with great care, and elaborate hydrographic and climatic observations have been made on them over a series of years. Long standing records of river flows under varying forest conditions have been scrutinized. Laboratory studies have been made, and volumes have been written, but to the present the only unchallenged truth that has been adduced is that the forest is a great consumer of water, which fact was already known. Forests consume from 15 to 30 in. of precipitation per year, and that is about all there is to it. If other effects exist, such as retarding the melting of snow, absorbing and holding moisture in the forest litter, stimulation of soil infiltration, and retarding of surface run-off, they are too insignificant to be of economic importance.

* "The Fight for Conservation," by Gifford Pinchot, Affiliate, Am. Soc. C. E., Doubleday, Page & Co., N. Y., 1910.

No one can study the masterly analysis* of this subject by the late H. M. Chittenden, M. Am. Soc. C. E., the reports of the Tenth Navigation Congress held at Milan, Italy, in 1905, the report† by D. W. Mead, M. Am. Soc. C. E., on Wisconsin conditions, the analysis of Merrimac and Connecticut River conditions by Edward Burr,‡ M. Am. Soc. C. E., and Harry Taylor,§ M. Am. Soc. C. E., the writer's paper entitled "Forests and Their Effect on Climate, Water Supply, and Soil,"|| and other papers and articles that treat this subject strictly on its merits, without being convinced that there is no foundation for the popular belief that reforestation is a practicable means of effecting stream regulation.

In water-sheds where the total precipitation is or will ultimately be required for irrigation and where the entire supply may be regulated by storage reservoirs, the forest on the water-shed is a detriment instead of a benefit, as far as water supply alone is concerned; because if it were not for the forest the total water yield would be increased by the amount the forest now consumes.

On the other hand, in regions of abundant rainfall the dense forests dissipate large quantities of the precipitation that would otherwise run off the area. This dissipation, however, is concurrent with the supply through increased evaporation and, during the growing season, through transpiration. Flood supplies are not held over to augment summer yields as is popularly believed. Forestation, therefore, as a means of water storage and stream regulation is without economic value from the standpoint of practical engineering.

CONCLUSIONS

In treating this subject generally no tangible means of effecting harmony between the use of water for power and irrigation has been disclosed. Both uses require artificial stream regulation, but irrigation having preferential rights, deprives power development of the full utilization of the supply that might otherwise obtain. Water is vitally indispensable for agriculture, but not for power, hence the justice of preferential use.

Regulation for power conserves surplus waters for use throughout the entire year, whereas regulation for agriculture conserves the surplus for use during about one-half the year. Generally speaking, greater storage capacities are required for complete utilization for irrigation than for power, and the shorter the growing season the greater the amount of storage required.

Uniformity of stream flow is a valuable characteristic in power development but loses much of that value when the entire supply is devoted to agriculture.

Complete regulation for both uses will require storage in high altitudes for power alone and foothill re-storage for irrigation. Power obtained from the utilization of released irrigation water will minimize the amount of power storage otherwise required.

* Transactions, Am. Soc. C. E., Vol. LXII (1909), p. 245.

† Bulletin No. 425, Univ. of Wisconsin.

‡ House Doc. No. 9, 62d Congress.

§ House Doc. No. 1294, 61st Congress.

|| Journal, Assoc. of Eng. Societies, July, 1913, p. 1.

Power from irrigation release has its most promising market in irrigation pumping, but the extent of this market is limited, and we can never hope to so utilize all the power that can thus be developed.

The problem of best utilizing the available water supply for both power and irrigation is strictly a local one and cannot be treated in general terms. Definite water supplies, definite storage opportunities, definite lands to be irrigated, and known power demands are essential prerequisites for its solution.

Primarily, actual regulation may be accomplished through the construction of storage reservoirs, and, secondarily, through soil storage, but not at all by reforestation.

In many cases harmony can be fostered and much can be done to eliminate conflicting interests. Power must concede the superior right to irrigation and in the long run will benefit substantially by that concession.

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TRANSACTIONS, AM. SOC. C. E., VOL. LXVII (1923), P. 245.
BULL. NO. 425, DIV. OF WATERSHEDS.
U. S. HOUSE DOC. NO. 254, 65th CONGRESS.
U. S. HOUSE DOC. NO. 1284, 61st CONGRESS.
JOURNAL, ASSOC. OF ENG. SOCIETIES, JULY, 1913, P. 1.

THE IMPROVED VENTURI FLUME

Discussion*

By RALPH L. PARSHALL, AFFILIATE, AM. SOC. C. E.†

RALPH L. PARSHALL,‡ AFFILIATE, AM. SOC. C. E. (by letter).§—When experiments were first made in the spring of 1915 on a type of measuring device having a converging and diverging section joined by a short throat section, with floor level throughout, this device was called the Venturi flume, because in plan it greatly resembled the converging and diverging sections as found in the Venturi meter, although in the flume the angles of convergence and divergence were equal. However, in this new device the water was not under pressure. As in the Venturi meter, two heads were observed, one in the converging and one in the throat section. Later, when experiments were conducted on the improved type of this device in which the angles of convergence and divergence were not equal and a depression in the throat section was introduced, this modified form was first called "the hydraulic jump flume"; but because of its similarity to the original type, both in design and section, it was thought advisable to designate it merely as the "improved Venturi flume."

For conditions of flow in which the discharge is a function only of the upper head and width of the throat, the law is not similar to that of the Venturi principle, but the device may be considered as a broad-crested weir without contractions. However, the additions of the depressed section at the throat introduces other conditions which modify its action. The improved Venturi flume, when under submergence of more than 70 to 75%, operates under the Venturi principle. It would seem, therefore, that it may or may not be a Venturi flume, according to the condition of flow.

From a theoretical standpoint, when the depth of water below the crest becomes positive, the flow is regarded as being submerged. As the depth below increases and approaches that of the upper depth, the percentage increases, approaching a limit when both these values are equal. Theoretically, under this condition the flow would be zero. For small differences or a high degree of submergence, when based upon readings taken as indicated in the plan and elevation of the improved Venturi flume, it has been found that a considerable discharge results when these heads are practically identical. The real effective head, however, is not considered because the points where the

* Discussion of the paper by Ralph L. Parshall, Affiliate, Am. Soc. C. E., continued from March, 1926, *Proceedings*.

† Author's closure.

‡ Senior Irrig. Engr., Bureau of Public Roads, U. S. Dept. of Agriculture, Fort Collins, Colo.

§ Received by the Secretary, February 24, 1926.

observations were taken do not give the true difference causing the velocity of the stream.

As pointed out by Mr. Lane,* the flow is theoretically submerged when the elevation of the water surface in the throat is higher than the crest elevation, but because the rate of discharge is not affected by this submergence until the ratio of heads approaches about 0.7 and because it follows the law of free flow, this range of conditions is called free flow and is a function only of the upper head and width of throat. When the upper head, H_u , is approximately 0.5 ft., the velocity of the water through the throat is sufficient to clear the throat section of the agitated condition, and the stream, through this section, is smooth and of relatively high velocity. When this condition is once established, the discharge may be reduced materially and yet this smooth condition of flow persists until the resistance, due to the upward inclined surface of the converging section, is sufficient to overbalance the momentum of the flowing stream. At this point the flow breaks and runs back into the throat. It is found that the law of flow is not changed where the stream passing the throat section is smooth or where the hydraulic jump is formed in the throat section, providing the ratio of the throat head to the upper head does not exceed approximately 0.7. Where the ratio of the throat head to the upper head exceeds this value of 0.7 the condition of flow is assumed to be submerged.

In his discussion, Mr. Carter† has developed a formula based on the original data limited in submergence from 69 to 79 per cent. The total free-flow data were not all presented in Table 2.‡ The data given were for the purpose of showing merely the agreement of computed and observed discharges for these limits of submergence.

In the determination of the free-flow discharge formula,

$$Q = 4 W H_u^{1.522} W^{0.026}$$

observations were considered only where the degree of submergence was not greater than 69.9 per cent. These data were obtained both at the Bellvue and Fort Collins laboratories for discharges ranging from 0.3 sec.-ft. to approximately 80 sec.-ft., the size of the flumes being 1, 2, 3, 4, 6, and 8 ft. These data, taken by various observers, were plotted logarithmically, giving the mathematical relation as stated.

The free-flow series consisted of 162 tests, of which 2 were rejected. The remaining 160 tests formed the basis for the determination of the free-flow expression previously stated. Of this number it was found that in the comparison of the computed and observed discharges, 155 tests, or 97%, agreed within plus or minus 3 per cent.

It is not claimed for the improved Venturi flume that its accuracy is superior to that of a standard weir, but from the writer's experience in the field with standard weirs, he would be inclined to think that the flume is as accurate as the ordinary weir under field conditions. This would be especially true where much sand or silt is carried in the stream.

* *Proceedings, Am. Soc. C. E.*, December, 1925, Papers and Discussions, p. 2030.

† *Loc. cit.*, January, 1926, Papers and Discussions, p. 104.

‡ *Loc. cit.*, September, 1925, Papers and Discussions, p. 1348.

The calibration of this type of measuring device of various sizes included an extensive series under submerged flow conditions. It appears from the study of these data that although the submerged flow does not give as close agreement as the free-flow condition, it may be assumed to be within practical limits. Of a total of 229 submerged flow tests, in which the upper head did not exceed 2.52 ft. and the degree of submergence ranged from 70 to 95%, 1 test was rejected. Of the remaining 228 tests, it was found that in the comparison of the computed and observed discharges, 194 tests, or 85%, agreed within plus or minus 5 per cent. The tests in which the submergence was greater than 95% were inconsistent and erratic and were not included in the computations.

In 1924 preliminary tests were conducted for the purpose of ascertaining the effects of introducing a parabolic curved surface as a floor in the throat section. These tests seem to indicate that an increase in the degree of submergence could be realized before it materially affected the free-flow discharge. The introduction of a warped surface, especially where structures are to be built by workmen of little mechanical experience, leads to the conclusion that the inclined plane floor surface in the throat section would be better, even if the effect of the degree of submergence would be slightly less. Tests were also made on a small sized improved Venturi flume having trapezoidal section.

It has been found from the practical application of the improved Venturi flume in the field, that protection is necessary below the structure when the stream is being discharged under free-flow condition. The exit velocities are relatively high, and unless the bottom and sides of the channel are protected, erosion will result. A wire net, filled with stone, anchored to the lower sill of the structure, has been found to be effective in preventing excessive erosion.

It would seem that the flume shown in Fig. 6* may be impractical because of the high velocities that would exist at the exit end, *E*; even when the section is expanded as in the improved Venturi flume, protection is frequently necessary. The measurement of the critical depth would, perhaps, be uncertain as well as difficult to obtain because of the doubt as to its location. To base the discharge on this one reading may not be altogether satisfactory, even if the simple relation between discharge and depth exists from theory. The structure shown in Fig. 6, from a practical standpoint, would be unduly expensive because of its great length, and it might be that an undisturbed flow in this section could not be depended upon. Too much weight should not be given to the advantage of its single gauge reading because submergence is always a possibility, and in that case it is necessary to have both the up-stream and throat-gauge readings to determine the actual discharge.

The type of flume shown in Fig. 16† will, in all probability, measure successfully under certain conditions but it is doubtful whether the head could be measured accurately at the critical depth. Experiments would be necessary to verify the statement that this device is immune from silting troubles. For small flows it would be expected that there would be an accumulation of deposit

* *Proceedings, Am. Soc. C. E.*, February, 1926, Papers and Discussions, p. 304.

† *Loc. cit.*, p. 317.

immediately up stream from the hump, and as the velocity increases this deposit may be washed out, thus changing the velocity of approach or contractions. Practical formulas of the type,

$$Q = L g^{\frac{1}{2}} D^{\frac{3}{2}}$$

will hold only when the critical depth is accurately determined. Since this point is unknown, it is not apparent how a formula of this type can apply without modification.

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The type of flume shown in Fig. 10† will, in all probability, measure accurately under certain conditions but it is doubtful whether the head could be measured accurately at the critical depth. Experiments would be necessary to verify the statement that this device is immune from silt and troubles. For small flows it would be expected that there would be an accumulation of deposit

*Proceedings Am. Soc. C. E. February, 1926, Issues and Discharge, p. 364.
†Loc. cit. p. 317.

SIDE CHANNEL SPILLWAYS: HYDRAULIC THEORY, ECONOMIC FACTORS, AND EXPERIMENTAL DETERMINATION OF LOSSES

Discussion*

By JULIAN HINDS, M. Am. Soc. C. E.†

JULIAN HINDS,‡ M. Am. Soc. C. E. (by letter).§—It is pleasing to note that the hydraulic principles proposed for the design of side channel spillways are generally accepted, with only a sufficient spirit of misgiving to insure the continued study of the problem. The explanations and improvements offered form a valuable addition to the paper.

The discussion by Mr. Muckleston|| of the "air rope", and his statements in regard to the effect of spiral flow, are interesting. From the statements made it might be inferred that the spiraling of the water around the axis of the channel, caused by the initial transverse velocity, is likely to increase the volume of flow materially. This probably should not be definitely accepted without proof. The statements in regard to the retention of air seem reasonable, and the spiral flow probably does tend to hold a part of the entrained air in the stream. However, it also promotes a rapid intermingling of particles, and tends to give a more nearly uniform axial velocity at any cross-section. A wide shallow channel would show a poor velocity distribution, which might more than offset the advantage resulting from the ready release of air. These spirals, or other equivalent trans-axial motions, are necessary for the temporary storage of the non-usable kinetic energy of the initial transverse motion, while it is being converted into heat. In a compact channel it is probable that the spiral action is more uniformly distributed to all the water particles, and is less violent. Under normal operating conditions there appears to be no tendency for the water to surge up the back wall of the channel.

The question raised is important and should be carefully considered in all future experimentation. Special attention should be given to "velocity swell" in the proposed laboratory tests in pipes. The fact that the mathematical treatment applies to the average velocity, and not to the mean velocity, as ordinarily computed, must be kept constantly in mind.

Mr. Wiley¶ also advocates a wide shallow channel, for mechanical reasons. A deep gash through the foundation adjacent to the end of the dam is without

* Discussion of the paper by Julian Hinds, M. Am. Soc. C. E., continued from February, 1926, *Proceedings*.

† Author's closure.

‡ Engr., Bureau of Reclamation, Denver, Colo.

§ Received by the Secretary, February 19, 1926.

¶ *Proceedings, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 2033.*

|| *Loc. cit., January, 1926, Papers and Discussions, p. 108.*

question objectionable. With a deep channel the structure supporting the spillway crest, which separates the channel from the reservoir, assumes formidable proportions, and an elaborate drainage system is required to avoid destruction of the channel lining by uplift. Some advantage may also result from the lower velocities that prevail in a wide channel. In addition a wide thin sheet of water is likely to be less destructive at the point of final discharge than a concentrated jet. Considered from this point of view, the channel should always be made as wide as conditions permit. However, if the hillside is steep the cost of a wide channel is likely to exceed greatly the cost of the protective measures necessary to make a deep channel safe. The cost of a wide shallow channel at a location, such as that existing at the Tieton Dam, on the Yakima Project, Washington, for example, would be prohibitive.

Apparently the method used by Mr. Wiley for computing the hydraulics of the channel will give results essentially identical with those obtained by the writer.

The partial list of side channel spillways given by Mr. Gutmann* is interesting. The Don Pedro Dam, in California, the Tieton Dam, in Washington, and the McKay Dam, now being constructed in Oregon, are notable additions to the list.

Mr. Gutmann's statement† that, in the preamble to Equation (17), $\frac{dA}{dA}$ is equal to dT only for a rectangular channel is in error, as the relation is general.

Mr. Houk and Mr. Howell both offer alternative plans designed to escape the losses involved in the side channel spillway. Mr. Howell‡ would use a deep, narrow, spillway notch, directed down stream, and controlled by a large Stoney gate. Mr. Howell's opinion that this is in all cases more economical than a side channel type is doubtless justified, and the advantages of a Stoney gate installation should be investigated wherever that type of control is considered adequate. The writer would not consider a Stoney gate spillway safe for the condition of a high earth dam in a heavily timbered region like that surrounding the Tieton Reservoir, for example.

Mr. Houk's suggestion§ that under favorable circumstances an advantage may be gained by curving the channel is believed to be sound. However, curvature of the main channel will not necessarily avoid impact loss, but by providing a means for reaction against the earth may make it possible to utilize a part of the energy of the transverse velocity.

It is possible that confusion sometimes results from the usual and convenient practice of speaking and thinking of eddies as causing losses of head. Under certain conditions it is impossible to maintain a balance between kinetic energy and static head without violating either the laws of motion or the law of conservation of energy. The situation is met by converting a part of the kinetic energy into heat. This transfer can not occur instantly, and

* *Proceedings, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 107.*

† *Loc. cit.*, p. 108.

‡ *Loc. cit.*, December, 1925, *Papers and Discussions, p. 2032.*

§ *Loc. cit.*, February, 1926, *Papers and Discussions, p. 319.*

the condemned kinetic energy is stored temporarily in transverse motions, which disappear as the water "warms up." The eddies are, thus, the result rather than the cause of the loss of head.

Mr. Stevens, near the end of his discussion,* makes the following statement:

"If the 'dead' water, or eddies, in sudden pipe enlargement or in a right-angle bend, can act as a diffuser then the eddies in a side channel spillway can perform the same function and all the energy of impact is not lost as the author assumes."

There are many secondary hydraulic problems in connection with side channel spillways which have not been solved, and for which no data are available. As pointed out in the paper, and in several of the discussions, there are many factors that require further study. However, there is no question as to the existence and approximate magnitude of the impact losses. To question the fact that all the energy of impact is lost is equivalent to questioning the validity of Newton's laws of motion. The mathematical application of the laws of motion to the particles composing a stream of water is simple, but a physical conception of the resultant reactions is difficult. Perhaps some simple illustrative device will be discovered before the study of impact losses is finally dismissed.

Mr. Stevens' statement that his Equation (34)† gives the eddy loss in a sudden enlargement, is not self-evident, and, so far as the writer knows, has never been rigidly proven. The so-called proofs usually given involve the assumption, or its equivalent, that the average unit pressure on the shoulder of the enlargement is equal to the unit pressure in the smaller pipe. The fact that under certain conditions the observed and computed losses do not agree is in all probability due to the inaccuracy of this assumption, rather than to any "diffusion" produced by "dead" water.

In discussing the testing of impact losses in pipes, Mr. Stevens proposes to combine the impact and sudden enlargement problems. This is believed to be undesirable. Either problem alone is sufficiently difficult. His Equation (27),† and all others dependent thereon, are based on the assumption that the pressure against the shoulder of the enlargement is equal to the pressure in the incoming pipes, the strict correctness of which is not apparent. It is believed that the enlargement should be eliminated from the tests, unless it is desired to extend the study to both these effects, which are distinct. It is believed that the tests at Berkeley, Calif., mentioned by Mr. Stevens, contemplate a main line pipe of uniform diameter.

The interest shown in this paper is gratifying. It is hoped that other discussions of the problem will be prepared for publication after the results of laboratory tests now under way are made public.

* *Proceedings, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 323.*

† *Loc. cit., p. 322.*

PERMISSIBLE CANAL VELOCITIES

Discussion*

By MESSRS. J. C. STEVENS, CARL ROHWER, AND IVAN E. HOUK

J. C. STEVENS,† M. A. M. Soc. C. E. (by letter).‡—The authors deserve the thanks of the Engineering Profession for their excellent paper. It summarizes briefly existing knowledge on the subject and offers its final conclusions in Table 6,§ which the writer believes will become the guide for earth canal design for years to come.

However, a word of caution should be sounded against using such data blindly. A thorough knowledge of the conditions to be met in design and of the materials encountered is essential to the proper interpretation of such guiding data. The authors have presented the results of their intimate knowledge and study of this subject in as nearly a "fool-proof" form as appears possible. Nevertheless, the terms "non-colloidal fine sands", "sandy loam and fine loam", "colloidal silts", etc., must give many engineers great concern. The skill of the chemist will be required in many instances.

It is obviously impossible to standardize soil classifications so that the various designations will have the same meaning for all. Broad experience will yet be a prime essential in order properly to interpret and successfully to utilize the data presented by the authors.

CARL ROHWER,|| Assoc. M. A. M. Soc. C. E. (by letter).¶—A knowledge of permissible canal velocities is an important factor in the design of irrigation systems, particularly where the soil is easily eroded, or the canals carry a large quantity of suspended material. Where both conditions obtain the velocity must be carefully determined to eliminate the possibility of erosion on one hand and sedimentation on the other. Fortunately, as pointed out by the authors, non-silting velocities are not necessarily scouring velocities.

An interesting example of a canal constructed under the conditions mentioned is shown in the view (Fig. 2), of the Fort Lyon Supply Canal in the Arkansas Valley near Rocky Ford, Colo. This canal is about 90 ft. wide, has a maximum capacity of 1500 sec.-ft., is excavated in Fresno fine sandy loam, and carries a large quantity of sand and colloidal silt. The silt remains in suspension in the canal but is deposited on the lands irrigated, amounting to

* Discussion of the paper by Samuel Fortier and Fred C. Scobey, Members, Am. Soc. C. E., continued from January, 1926, *Proceedings*.

† Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

‡ Received by the Secretary, December 19, 1925.

§ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1411.

|| Associate Irrig. Engr., Div. of Agri. Eng., U. S. Dept. of Agriculture, Colorado Experiment Station, Fort Collins, Colo.

¶ Received by the Secretary, December 21, 1925.

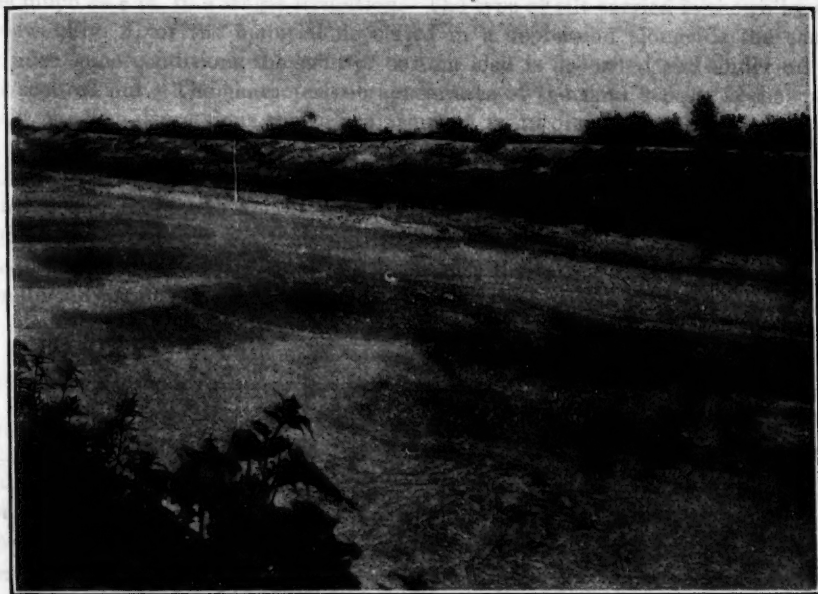


FIG. 2.—FORT LYON SUPPLY CANAL, NEAR ROCKY FORD, COLO., AUGUST 13, 1924, SHOWING ERODED AREA.

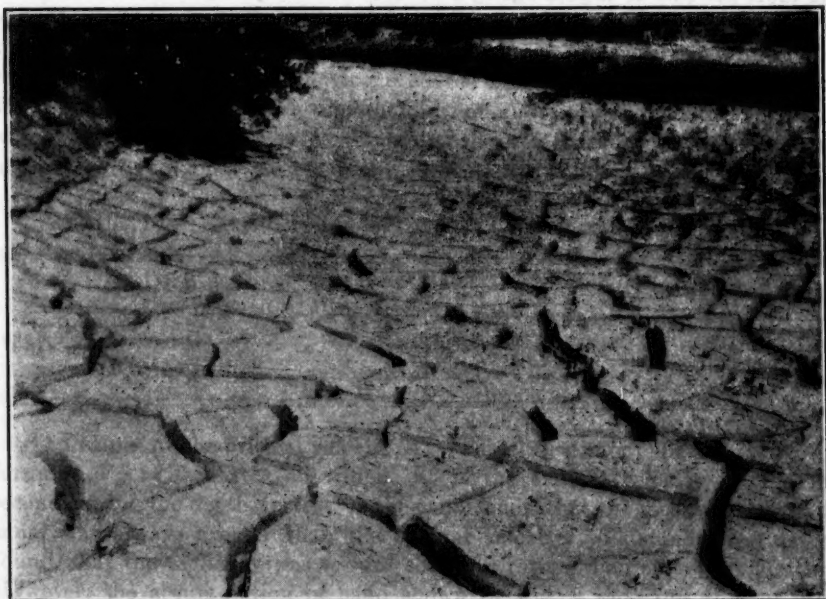


FIG. 3.—SILT DEPOSIT IN DEPRESSION ALONGSIDE FORT LYON SUPPLY CANAL, NEAR ROCKY FORD, COLO., AUGUST 13, 1924.



FIG. 2.—View from St. Louis, Mo., looking down the Mississippi River, showing the power plant and the canal project.



FIG. 3.—View from St. Louis, Mo., looking down the Mississippi River, showing the power plant and the canal project.

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as much as $\frac{1}{4}$ in. in a single irrigation. The type of silt carried is shown by the view (Fig. 3) of the material deposited in a depression alongside the canal. Under some conditions the sand or bottom load is deposited and under others is scoured out. The characteristic appearance of the sand deposit is shown in Fig. 2. The depressions shown are about 15 ft. in diameter and 3 ft. deep.

Tables 9 and 10, submitted by Mr. H. D. Amsley, of the State Engineer's Office, give the flow characteristics of the canal based on current meter measurements and the gauge height record at the rating flume near the head. The quantities given are for the conditions in the rating flume, but they may be assumed to apply to the canal proper because the rating section does not differ materially from the canal section.

TABLE 9.—DISCHARGE MEASUREMENTS ON FORT LYON SUPPLY CANAL.

Date.	Gauge height, in feet.	Area, in square feet.	Discharge, in cubic feet per second.	Mean velocity, in feet per second.	Critical velocity, V_0^* , in feet per second.
June 8, 1923.....	3.44	304.5	681.5	2.24	1.85
" 14, ".....	1.82	101.5	159.4	1.57	1.00
" 19, ".....	4.96	452.7	1 142.5	2.52	2.35

* Computed from the formula, $V_0 = 0.84 d^{0.64}$.

On June 8, 1923, the depth in the flume for the gauge height, 3.44 ft., varied from 2.02 to 3.44 ft., and on June 19, for a gauge height of 4.96 ft., from 4.90 to 5.00 ft. This shows that more than 1.25 ft. were scoured from some parts of the flume in that interval, and as the conditions in the canal are similar it is reasonable to assume that scouring occurred there also. During this time, as shown in Table 10, the gauge height did not materially exceed the values for which the discharge is given, therefore, the velocities were probably never materially higher than the maximum indicated in Table 9.

TABLE 10.—MAXIMUM GAUGE HEIGHTS, FORT LYON SUPPLY CANAL.

Date.	Hour.	Gauge height, in feet.
June 8, 1923.....	4 A.M.	4.75
" 16, ".....	6 P.M.	5.15
" 19, ".....	9 A.M.	5.00

The values of the critical velocity, V_0 (Table 9), computed from Kennedy's formula, $V_0 = 0.84 d^{0.64}$, in which, V_0 is the critical velocity, in feet per second, and d , the depth, in feet, of the canal, show that the mean velocity of the canal was always greater than the critical velocity. This indicates, as was found, that the tendency was toward scouring rather than silting at the time. However, the accumulation of sand shown in Fig. 2 indicates that at other times considerable sand is deposited, but the established regimen of the canal indicates that from season to season as much sand is scoured out as is deposited.

IVAN E. HOUK,* M. AM. SOC. C. E. (by letter).†—The paper presents several good ideas and many valuable data, and the authors are to be congratulated on the clear and comprehensive manner in which they have handled the subject. Their discussion of the effect of colloidal silts in increasing scouring velocities is both interesting and valuable; and the practical ideas and data,‡ submitted by engineers in charge of irrigation projects, should prove of great value in future designing work. The velocities to be permitted in canals are primarily dependent on the nature of the material through which the canals are located, a condition which is susceptible of almost infinite variation. It is believed that, with the present knowledge and available data, detailed descriptions of the materials through which certain canals have been built, together with the actual velocities found feasible therein, are more valuable than generalized recommendations, such as are given in Table 6.§

It is not clear to the writer where there has been any confusion, fallacy, or fundamental error in the work and discussions of other engineers, particularly the British irrigation engineers who have had long and valuable experience in such matters. They have frequently pointed out a wide difference between non-silting and scouring velocities. That they have discussed deposition and erosion in a more or less combined manner, is true; but it seems perfectly logical to do this. In designing a canal to be located through extensive silt deposits it is important to adopt velocities that will neither silt nor scour; and it is perfectly proper to consider such phenomena at the same time even if they are fundamentally different operations. However, experimental determinations of non-silting and eroding velocities are matters which undoubtedly should be considered separately.

It is believed that the experiments, experience, researches, and discussions of many Eastern irrigation engineers, including Kennedy, Willcocks, Buckley, Parker, Bellasis, and others, are well worthy of careful study by any one interested in the problem. Although their experience may not agree exactly with that of Western engineers, it should not be so briefly dismissed. It is believed that the difference between their experience and that of irrigation engineers in the United States is due to the differences in the materials through which the canals are located, rather than to any fallacy or fundamental error in their conclusions. Their studies and experiments, in some cases at least, may have been made on canals where no colloidal silt was present. There probably are many locations in the United States where Kennedy's laws are more applicable than the recommendations of the authors. The writer knows of several power canals in the East where the velocities recommended by the authors could not be permitted.

It might also be pointed out that the effect of the channel depth is not considered in the final recommendations of the authors. Although the writer has never attempted to express his observations by formulas, he has

* Engr., U. S. Bureau of Reclamation, Denver, Colo.

† Received by the Secretary, February 2, 1926.

‡ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, pp. 1404-1407.

§ *Loc. cit.*, p. 1411.

TABLE 11.—SUMMARY OF SCOUR OBSERVATIONS IN
 THE GREAT MIAMI VALLEY.

Location.	Maximum average velocity, in feet per second.	Mean depth, in feet.	Material.	Results.
Overflow area of Miami River, Miamitown, Ohio.....	11.5	24.6	Alluvial soil.	No evidence of scour.
"Big Four" Railroad bridge over Buck Creek, Springfield, Ohio.....	13.7	6.7	Clay, sand, and gravel, with some loose rock around abutments.	No evidence of scour.
Limestone Street Bridge over Buck Creek, Springfield.....	10.1	11.5	Clay, sand, and gravel, with some loose rock around abutments.	No evidence of scour.
"Big Four" Railroad bridge over the Miami River, Sidney, Ohio.....	12.4	15.7	Earth and gravel with probably rock in river bed.	Scour around east end of bridge and south side of embankment. None in river bed.
Miami River above the Cincinnati Hamilton and Dayton Railroad bridge, Troy, Ohio...	6.11	18.6	Clay, sand, and gravel with some broken rock.	No evidence of scour.
Miami River at the Cincinnati Hamilton and Dayton Railroad bridge, Troy.....	11.1	16.3	Clay, sand and gravel with some broken rock.	Considerable scouring around piers.
Overflow areas of Miami River, below Miamisburg, Ohio.....	6.67	16.7	Alluvial soil.	No evidence of scour.
Miami River Bridge, Tadmor, Ohio.....	7.8	About 23	Sand and gravel with some broken rock.	No evidence of scour.
Miami River channel below Miamisburg.....	9.23	31.1	Clay and gravel.	No evidence of scour.
"Big Four" Railroad bridge over Miami River below Miamisburg.....	15.2	27.7	Clay, gravel, and rock.	Extensive scour.
Stillwater River above highway bridge, West Milton, Ohio....	12.0	27.7	Clay and gravel.	No evidence of scour.
Stillwater River at highway bridge, West Milton.....	14.6	26.3	Clay and gravel with broken limestone around abutments.	Extensive scour.
Mad River at first "Big Four" Railroad bridge west of Springfield.....	12.2	17.2	Clay and gravel with some bed-rock in middle of channel.	No evidence of scour.
Mad River at second "Big Four" Railroad bridge west of Springfield.....	15.0	18.0	Clay and gravel with some bed-rock in middle of channel.	Extensive scour.
Miami River at Main Street Bridge, Dayton, Ohio.....	11.0	*24.6	Gravel.	Considerable scouring around piers.
Miami River overflow above Tadmor.....	5.87	13.3	Alluvial soil.	No evidence of scour.
Miami River channel above Tadmor.....	8.17	23.4	Gravel.	No evidence of scour.
Miami and Erie Canal opening at Miami River below Tadmor.	16.6	30.6	Gravel and rock.	Extensive scour.
Miami and Erie Canal stone arch opening at Turtle Creek, west of Sidney, Ohio.....	13.7	10.1	Clay and gravel.	Some scour.
"Big Four" Railroad concrete arch opening at Tawawa Creek, east of Sidney.....	23.1	8.2	Clay and gravel.	Scour.
Seven Mile Creek, Seven Mile, Ohio.....	5.44†	2.58	Limestone.	No evidence of scour.
Miami River, Sidney.....	3.90†	5.06	Gravel and rock.	No evidence of scour.
Miami River, Tadmor.....	4.63†	9.85	Gravel.	No evidence of scour.
Miami River at Main Street, Dayton.....	4.54†	10.1	Gravel.	No evidence of scour.
Miami River just above Wolf Creek, Dayton.....	5.0†	5.2	Gravel.	Slight shifting of material.
Stillwater River, West Milton...	6.36†	8.70	Gravel.	No evidence of scour.
Miami River, Dayton.....	2.68†	Gravel.	No scour.
Miami River cut-off channel, Dayton.....	5.5†	1.3	Gravel.	No scour.
Miami River cut-off channel, Dayton.....	5.5†	1.5	Clay.	Continuous erosion.
Miami River cut-off channel, Dayton.....	6.1†	3.2	Gravel.	Extensive scour.

* Average depth under bridge.

† Velocity measured with a current meter.

observed, and always considered, that higher velocities can be permitted in deep channels than in shallow channels, as has been maintained by many writers. On the Miami Valley flood prevention work at Dayton, Ohio, velocities as high as 10 ft. per sec. were considered allowable as extreme maximum values in straight channels having gravel or cobble beds, where the depths were 20 ft., or more. Of course, such high velocities could not be permitted in irrigation canals through similar materials. They would not have been considered on the Miami Valley work except as ultimate possible limits to occur only for a few hours' duration, but even under such conditions the writer would not consider them safe for channels 5 ft. deep. In curved channels on the Miami Valley work concrete slab paving was placed on the outer banks, and flexible concrete block mattresses on the bottom, adjoining the outer banks, wherever the mean velocity exceeded 6 ft. per sec.*

In the high-water surveys of the 1913 flood in the Miami Valley,† made during the summer after the flood, it was noticed that unusually high velocities can occur over alluvial soils when the depths are comparatively great and the flow comparatively uniform, without causing appreciable scour. Several instances were observed where mean velocities as high as 6 ft. per sec. occurred over alluvial fields along the main channels, with mean depths of about 15 ft., without causing any noticeable erosion. Cornstalks from the preceding year's crop were still in place after the flood had passed. In one instance a velocity estimated at 11.5 ft. per sec. occurred over fields flooded to a mean depth of 24.6 ft., where the flow conditions were unusually uniform, without causing apparent scour. Immediately below, where the velocities were practically the same, but the flow conditions greatly disturbed, a new channel about 100 to 150 ft. wide and about 10 ft. deep, was eroded. The material in both places consisted of an alluvial surface soil underlaid by a yellow clay containing some sand and gravel.

Although the velocities just mentioned were calculated from surveys made after the flood, they are not believed to be appreciably in error. The surveys were carefully made, and the calculations, using the Kutter formula, were checked by independent computations based on measured head at contracted openings, as described in full in the report previously noted. The more valuable observations, together with some later data obtained where the velocities were measured with a current meter, are given in Table 11. Velocities measured with a current meter are indicated by asterisks.

The observations on the Miami River cut-off channel, which make up the last three items in Table 11, are probably worth some discussion. Fig. 4 is a view of the cut-off looking up stream along the channel, from a point near the lower end. The material forming the bed and sides of the cross-section above the place where the man is standing was clean sand and gravel; below this place it was a tenacious yellow clay. Fig. 5 is a close view of the gravel, taken on the berm which appears at the left of Fig. 4. The folding rule and engineer's scale indicate the sizes of the particles. Although 2 to 3-in. stones

* "Hydraulics of the Miami Flood Control Project," by Sherman M. Woodward, Technical Reports, Pt. VII, The Miami Conservancy District, Dayton, Ohio, 1920.

† "Calculation of Flow in Open Channels," by Ivan. E. Houk, Technical Reports, Pt. IV, The Miami Conservancy District, Dayton, Ohio, 1918.

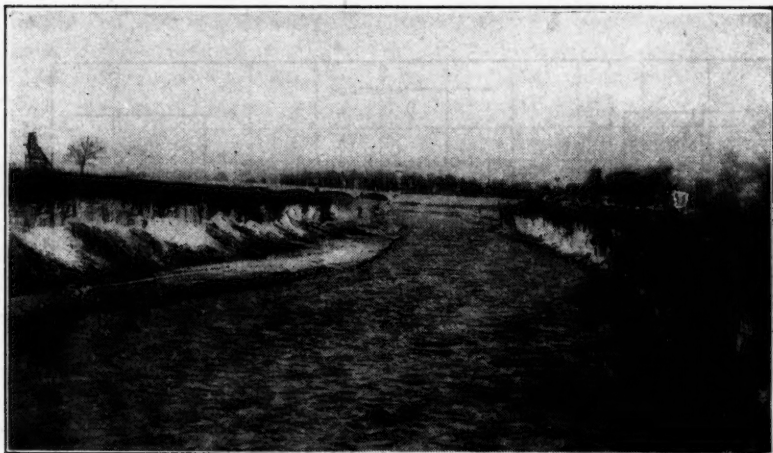


FIG. 4.—VIEW OF MIAMI RIVER CUT-OFF, LOOKING UP STREAM ALONG THE CHANNEL.

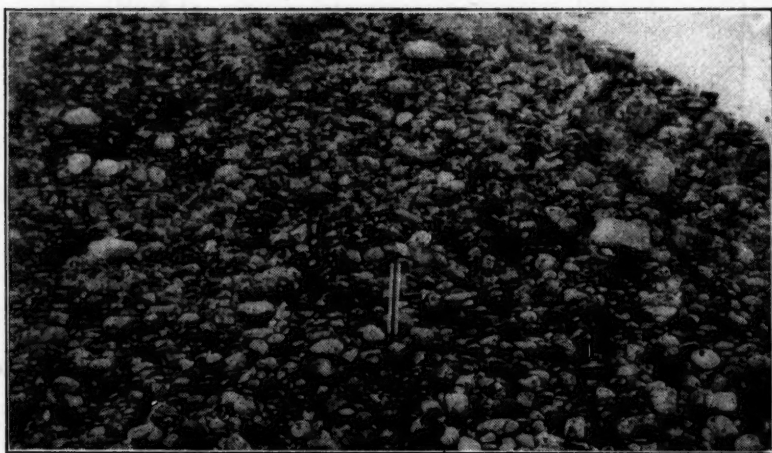
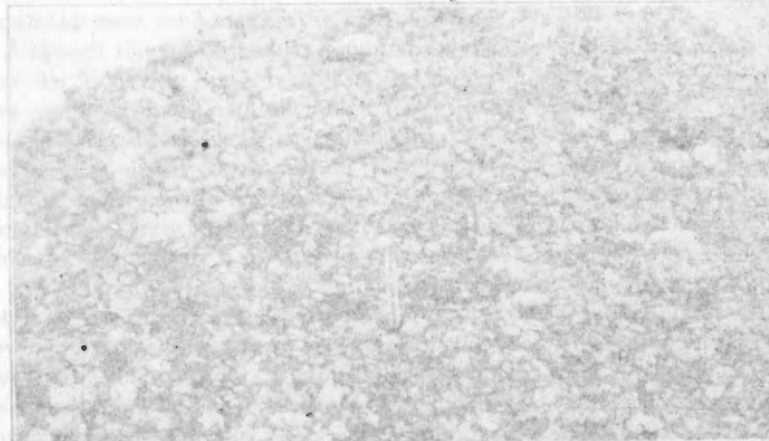


FIG. 5.—GRAVEL FORMING BED AND BANKS OF THE MIAMI RIVER CUT-OFF.

It is believed that the velocity of the water in the canal is not uniform, but that it varies in different parts of the canal. The velocity is greatest in the middle of the canal, and least near the banks. The velocity is also greater in the upper part of the canal, and less in the lower part. The velocity is also greater in the middle of the canal, and least near the banks. The velocity is also greater in the upper part of the canal, and less in the lower part.



The view of the canal from the bridge is very fine. The water is very clear, and the banks are very green. The sky is very blue, and the sun is shining brightly. The view is very beautiful, and it is a pleasure to look at it. The view is very beautiful, and it is a pleasure to look at it. The view is very beautiful, and it is a pleasure to look at it.



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constitute the greater part of the surface material, finer particles, fairly well graded, were beneath the surface. This material seemed to be fairly typical of the river-bed gravel found in other parts of the Miami Valley.

When the first set of current-meter measurements were taken, showing a mean velocity of 5.5 ft. per sec., the clay section was eroding continuously, whereas the gravel section was stable. The gravel seemed to be fast on the point of moving, but no sand or pebbles were being transported in suspension

and no movement along the bottom could be detected. Fortunately, the clay section was higher than the gravel section so that the water above the clay was clear. When the second set of measurements was taken, four days later, the gravel section was still stable, but the clay section was eroding more rapidly, and the water was muddy. The sounding showed only a short distance to the water.

That it was the clay which was being eroded was proved by the fact that the sounding showed only a short distance to the water. The height of the gravel section was 5 and 6 ft. per sec. probably near the water level, and the height of the clay section was 5 and 6 ft. per sec. probably near the water level.

On the basis of the above, the writer believes that the main limiting factor in the design of straight channels where the beds are "scoured" and "collected" is the velocity of the water. The velocity of the water should be such that the water will be fast enough to move the material which is being eroded, but not so fast that the material will be transported in suspension.

On the basis of the above, the writer believes that the main limiting factor in the design of straight channels where the beds are "scoured" and "collected" is the velocity of the water. The velocity of the water should be such that the water will be fast enough to move the material which is being eroded, but not so fast that the material will be transported in suspension.

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FIG. 6.—CROSS-SECTIONS OF MIAMI RIVER CUT-OFF CHANNEL, DAYTON, OHIO, SHOWING SCOUR CAUSED BY INCREASE IN VELOCITY FROM 5.5 TO 6.1 FEET PER SECOND.

constitute the greater part of the surface material, finer particles, fairly well graded, were beneath the surface. This material seemed to be fairly typical of the river-bed gravel found in other parts of the Miami Valley.

When the first set of current-meter measurements were taken, showing a mean velocity of 5.5 ft. per sec., the clay section was eroding continuously, whereas the gravel section was stable. The gravel seemed to be just on the point of moving, but no sand or pebbles were being transported in suspension and no movement along the bottom could be detected. Fortunately, the clay section was down stream from the gravel section so that the water above the clay was perfectly clear. When the second set of meter measurements was taken, four days after the stable conditions had been changed by a slight rise in the river, the mean velocity was 6.1 ft. per sec., only 0.6 ft. per sec. greater; but the gravel was being rapidly eroded. Pebbles of considerable size were being carried in suspension near the surface of the water, as could be determined by placing the sounding rod only a short distance into the water. Thus, it was definitely proved that this material began to erode at some velocity between 5.5 and 6.1 ft. per sec. probably nearer the lower value, when the depths were between 2 and 3 ft. The shaded portions of the cross-sections in Fig. 6 indicate the total cross-sectional area eroded during the four days. Taking into account the length of the cut-off, about 600 ft., rough calculations show that approximately 100 cu. yd. per hour were being transported.

On the basis of the cut-off experiments the writer believes that the maximum permissible velocities of 4.00 and 5.00 ft. per sec., given in Table 6 for limiting values to be used in straight channels where the beds are "coarse gravel" and "cobble and shingles", respectively, and where the water is clear, could be raised to 5.0 and 5.5 ft. per sec., respectively, especially since very few canals designed for such velocities will be less than 2 or 3 ft. deep. For large canals, 10 or 15 ft. deep, the writer believes that still higher velocities can sometimes be permitted, possibly 7 or 7.5 ft. per sec. in emergencies. What really happens in a canal located through gravelly materials, subjected to relatively high velocities, is that the finer particles are washed out until the bottom is literally paved with the larger stones; so that the larger stones protect the finer particles, below, from further scour.



MOMENTS IN RESTRAINED AND CONTINUOUS BEAMS BY THE METHOD OF CONJUGATE POINTS

Discussion*

BY MESSRS. JOHN J. GUT, RICHARD G. DOERFLING, R. McC. BEANFIELD,
AND WALTER RUPPEL

JOHN J. GUT,† Assoc. M. Am. Soc. C. E. (by letter).‡—The graphical solution of continuous beams and frames as presented by the authors has brought to engineers a considerable simplification in structural design.

In the past few years, in Europe, extensive attention has been given to the graphical solution of reinforced concrete frames. Among others, the books published by Strassner§ have treated this subject in a very thorough and complete manner.

To those not familiar with German literature, a short description of the principles and methods used by Strassner as compared with those of the authors, may demonstrate the value of the new method. Strassner applied to the design of frames Mohr's principles|| that:

- (1) The angular deflection of the elastic line at the supports is equal to the reaction of the beam loaded by its moment-area divided by EI ; and
- (2) The ordinates of the elastic line are equal to the bending moment of the beam loaded by the moment-area divided by EI .

Strassner, using Ritter's graphical method of finding the points of inflection of the elastic line (of unloaded spans), determines, by means of these points and Mohr's principles, the negative moments of the actual loaded beam. The application of these principles may be best illustrated by the following example:

Fig. 58 shows a frame with a single concentrated load. The dotted lines indicate deflection lines which are assumed for a preliminary determination of moments and shears.

Fig. 59 represents the moments resulting from a unit horizontal load acting at the upper left corner of the frame and is used for correcting the "preliminary" moments and shears as shown on Figs. 61 and 62, which are self-explanatory.

* Discussion of the paper by L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., continued from March, 1926, *Proceedings*.

† San Francisco, Calif.

‡ Received by the Secretary, January 12, 1926.

§ "Berechnung statisch unbestimmter Systeme," Bd. I-II; "Neuere Methoden," Bd. I-II.

|| A detailed demonstration of these principles has been given by Charles S. Whitney, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 122 et seq.

Fig. 59 also shows how the *A*-points, the points of zero moment, are obtained. Point J_1 is projected to the left and Point K_1 to the right. Then Line 1 is drawn, followed by Lines 2 and 3. Point K_3 is projected to the left and Point J_3 to the right. Then Line 4 is drawn, followed by Lines 5 and 6. Lines 7, 8, and 9 represent the sum of Lines 1 to 6, inclusive.

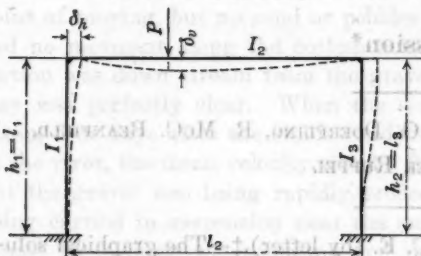


FIG. 58.—FRAME WITH SINGLE CONCENTRATED LOAD.

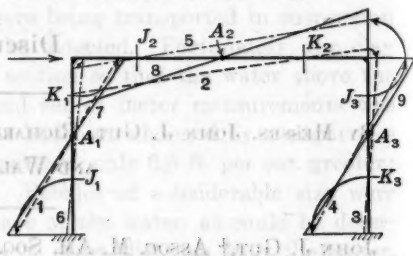


FIG. 59.—DETERMINATION OF A-POINTS.

Fig. 60 shows how the points of inflection—fixed points—or *J* and *K*-points are obtained. After locating the *V* and *U*-lines at the third points of the span and the *T*-line, as illustrated, Line 1 is drawn at any angle. Then Lines 2, 3, 4, 10 are successively drawn in the order named.

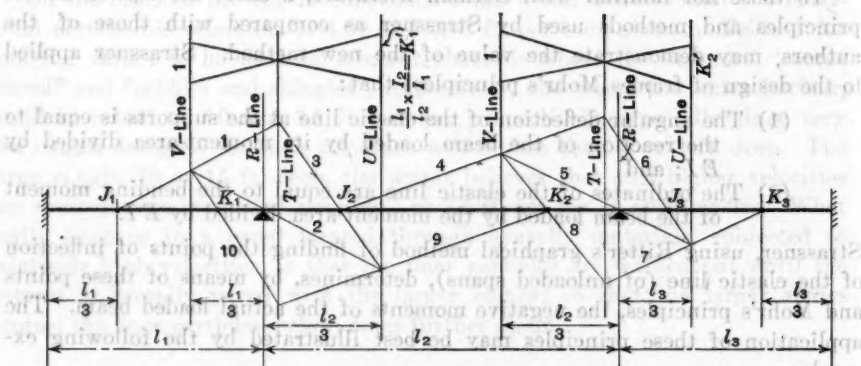


FIG. 60.—DETERMINATION OF *J* AND *K*-POINTS.

Due to the unsymmetrical load conditions the shears, S_1 and S_2 , obtained from the moments shown on Fig. 61 are not equal. For these conditions the frame would not be in equilibrium and a horizontal displacement equal to δ_h (Fig. 58) would take place. Fig. 63 illustrates how Strassner corrects, by a very simple method, these moments (as shown on Fig. 61), making thereby $S_1 = S_2$ (Fig. 62).

Draw any horizontal line, 1 (Fig. 63). Then draw parallel to Lines M_1 and M_2 (see Fig. 61) Lines M_1' and M_2' . Locate Point *G* perpendicular to and in the center of Line 1. Then Lines 2 and 3 give the new inclinations of the column moment lines based on the condition that the shears, S_1 and S_2 , are equal.

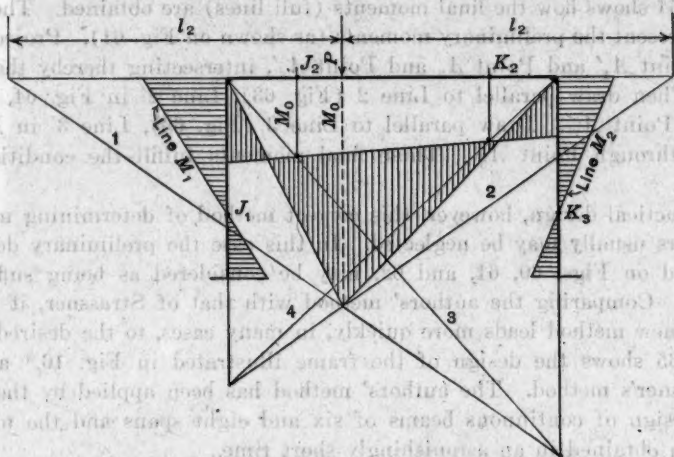


FIG. 61.—DETERMINATION OF PRELIMINARY MOMENTS.

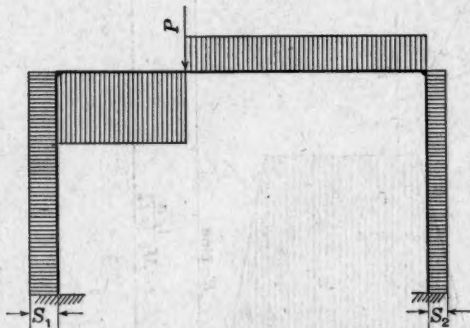


FIG. 62.—PRELIMINARY SHEARS.

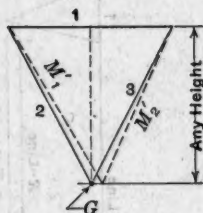


FIG. 63.

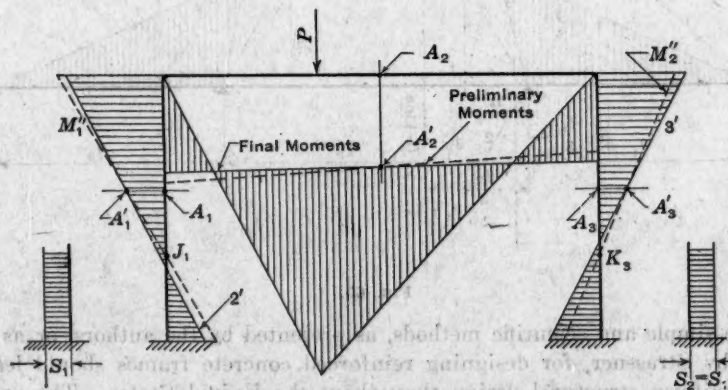


FIG. 64.—DETERMINATION OF FINAL MOMENTS AND SHEARS.

crete, as specified in most of the building codes, are irrational and involve an unnecessary and wasteful financial burden. Scientific design of a concrete structure not only is more economical, but also often safer. Numerous tests have confirmed plainly the fundamental assumptions of the science.*

An interesting comparative design between a 6-story reinforced concrete building in Los Angeles, Calif., and the same building as it would have been built in San Francisco, Calif., has been made by the writer. The Building Code of San Francisco gives the engineer the privilege of using scientific principles, specifying in Part VIII, "Design in General, Restrained Beams", that:

"All continuous and restrained slabs, beams, and girders shall be designed to resist, at all supports, the full moments and between all supports, $\frac{3}{4}$ of the moments resulting from all spans being fully loaded. In no case, however, shall the resisting moments between supports be less than bending moments produced by any condition of partial loading."

For the building in question girders 30 in. deep had to be used in Los Angeles, whereas in San Francisco girders 24 in. deep would have been sufficient to provide for the actual stresses. From the economic point of view a reduction of 3 ft. in building height in this case, as well as greater safety and a material additional saving in cost, justify plainly a thoroughly scientific and rational design.

The writer believes that the authors' method, or any other quick method, is of primary importance for a sane and economic development of reinforced concrete construction. The scientific value of the method merits the attention of university courses throughout the United States.

RICHARD G. DOERFLING,† M. Am. Soc. C. E. (by letter).‡—The writer has read this paper with much interest and is pleased to note that the Society has given free scope to papers like those by Cecil Vivian von Abo, Jun. Am. Soc. C. E.,§ Charles S. Whitney, M. Am. Soc. C. E.,|| and this one.

For the most part the methods demonstrated in these papers were invented and developed from 30 to 60 years ago in Switzerland, Austria, and Germany, where they have become the working tools of structural engineers. Although they occupy considerable space in German technical literature, they are not readily accessible in English. In fact, what little has entered English literature has come in rather timidly and fragmentarily, and remained almost unknown after facing some stern critics who boldly doubt the existence of secondary stresses, do not agree that a concrete arch should be treated as an elastic structure, and would remedy continuous beams by replacing the elusive fixed points with substantial joints that can be seen and inspected, and oiled if necessary.

* "Concrete—Plain and Reinforced," by Taylor, Thompson, and Smulski, 1925, p. 64, etc.; also, "Vorlesungen über Eisenbeton," by E. Probst.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, January 14, 1926.

§ "Secondary Stresses in Bridges," *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, p. 969.

|| "Design of Symmetrical Concrete Arches," *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 931.

A perplexing question is frequently raised as to "what is the best and shortest method of doing a certain thing", which rather involves the personal equation. The best method of course for any one person, and also the shortest and safest method for him, is the one he understands best and feels sure about, be it graphical, analytical, or a combination of these methods.

This paper furnishes useful graphical methods for many problems of continuous beams and frames. The authors have preferred the roundabout way of representing the methods demonstrated as analytical deductions from the geometry of graphics again translated into graphics, while the simpler and clearer proof may be furnished by the geometry of the construction directly, in most cases by similar triangles or ratios.

The "conjugate points" of the authors are readily recognized, by those acquainted with the literature on the subject, as the projections, F' , of the fixed points, F , upon the M -polygon. The name and designation only are new, and the derivation in reversed order—rather a retrograde step in regard to clearness in demonstrations as well as in application. The multiplicity of points and lines in the body of the diagram is confusing (see Figs. 11* or 13,† for instance), and in practical application the lines will frequently intersect at angles too acute for precise determination of other points and lines required. On the other hand a random construction for the fixed points outside the body of the diagram obviates both these objections, the intersection angles can be chosen at will.

Arbitrary coefficients in building ordinances giving moments 50% greater or less than the true moments for uniformly distributed loads are not altogether wrong, considering the fact that the live load on adjacent floor-panels may in reality vary from zero to the maximum, upsetting all fine-spun computations for uniform loading on continuous beams and slabs. The sharp peak moments over concrete columns and piers could be safely rounded off parabolically to one-half within the width of the column (because these supports are distributed and not knife-edges), while the moments at mid-span should be increased to guard against unbalanced loads.

Although the more laborious means of influence lines show most clearly how the live load on continuous beams must be distributed for maximum moment at any point, the next best method, and one very appropriate for more important work in building construction, is to determine the moments with the load on one span only, and add the results, either graphically or arithmetically. This can be done by very simple methods—simple in demonstration as well as in application, with little tax on the memory. In the belief that a presentation of these methods will shed more light on the important elements and characteristics of the whole subject, the writer offers the following contribution as a supplement to the paper, and, incidentally only, makes his demonstration bear out his critical contention. Some repetition will be unavoidable.

(1).—*Elastic Weights*.—As an introduction it seems pertinent to discuss briefly the subject of "elastic weights".

* *Proceedings, Am. Soc. C. E.*, October, 1925, Papers and Discussions, p. 1600.

† *Loc. cit.*, p. 1602.

The maximum bending moment, M , caused by a load, W , is directly proportional to W and to the distance, l , at which W acts.

The maximum deflection, D , caused by a bending moment, M , is directly proportional to M and to the distance, l , at which M acts, and inversely proportional to the stiffness, $E I$, of the structural member under consideration.

From this analogy $M = l W$ and $D = l \frac{M}{E I}$. The term, $\frac{M}{E I}$, is called the elastic weight, and deflections and deflection diagrams are determined (like moments and moment diagrams) by using the moment area as a load area and dividing by $E I$. Assume a force diagram and an equilibrium polygon constructed with a pole distance equal to $E I$; then D at any point is the intercept of the equilibrium polygon at that point.

In general: Using the load area,

$$M = \text{intercept} \times \text{pole distance}$$

and using the moment area as a load area,

$$D = \frac{\text{intercept} \times \text{pole distance}}{E I}$$

In order to gain a working acquaintance with the fundamental idea of elastic weights, deflection formulas may be derived by its use for the following four familiar cases:

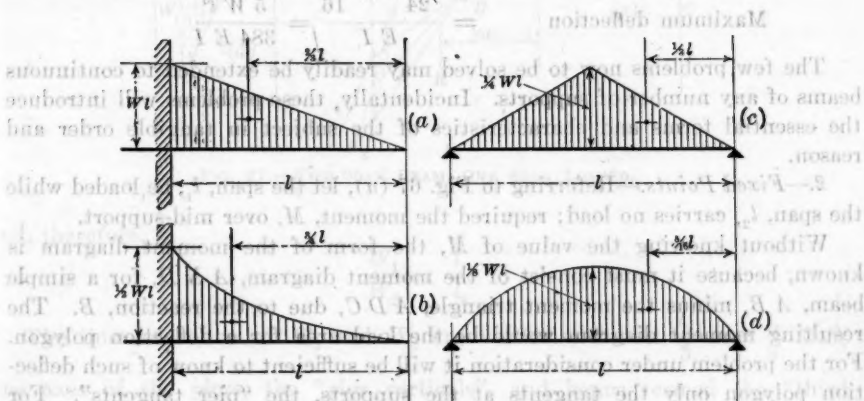


FIG. 66.—DEFLECTIONS FOR TYPICAL LOADING BY THE METHOD OF ELASTIC WEIGHTS.

Cantilever with a load, W , at the end (Fig. 66 (a)):

Maximum moment

$$= W l$$

Moment area

$$= W l \times \frac{l}{2} = \frac{W l^2}{2}$$

Maximum deflection

$$= \frac{\frac{W l^2}{2} \times \frac{2 l}{3}}{E I} = \frac{W l^3}{3 E I}$$

Cantilever with a load, W , uniformly distributed (Fig. 66 (b)):

$$\text{Maximum moment} = \frac{Wl}{2}$$

$$\text{Moment area} = \frac{Wl}{2} \times \frac{l}{3} = \frac{Wl^2}{6}$$

$$\text{Maximum deflection} = \frac{\frac{Wl^2}{6} \times \frac{3l}{4}}{EI} = \frac{Wl^3}{8EI}$$

Simple beam with a load, W , at the center (Fig. 66 (c)):

$$\text{Maximum moment} = \frac{Wl}{4}$$

$$\text{One-half the moment area} = \frac{Wl}{4} \times \frac{l}{4} = \frac{Wl^2}{16}$$

$$\text{Maximum deflection} = \frac{\frac{Wl^2}{16} \times \frac{l}{3}}{EI} = \frac{Wl^3}{48EI}$$

Simple beam with a load, W , uniformly distributed (Fig. 66 (d)):

$$\text{Maximum moment} = \frac{Wl}{8}$$

$$\text{One-half the moment area} = \frac{Wl}{8} \times \frac{l}{3} = \frac{Wl^2}{24}$$

$$\text{Maximum deflection} = \frac{\frac{Wl^2}{24} \times \frac{5l}{16}}{EI} = \frac{5Wl^3}{384EI}$$

The few problems now to be solved may readily be extended to continuous beams of any number of supports. Incidentally, these problems will introduce the essential terms and characteristics of the subject in tangible order and reason.

2.—*Fixed Points*.—Referring to Fig. 67 (a), let the span, l_1 , be loaded while the span, l_2 , carries no load; required the moment, M , over mid-support.

Without knowing the value of M , the form of the moment diagram is known, because it must consist of the moment diagram, AND , for a simple beam, AB , minus the moment triangle, ADC , due to the reaction, B . The resulting moment diagram would be the load area for a deflection polygon. For the problem under consideration it will be sufficient to know of such deflection polygon only the tangents at the supports, the "pier tangents". For determining these tangents the load area, Fig. 67 (a), is considered to be composed of three parts, the area, $AND = W_0$, termed the "simple moment area", and the triangles, $ABD = W_1$ and $CBD = W_2$. These three areas are used as elastic weights acting at the gravity lines of the areas. The first is positive, causing downward bending, and the other two are negative, causing upward bending. The numerical value of W_0 , the simple moment area, may be computed, and it is seen that the ratio of W_1 and W_2 remains constant whatever the value of M , for

$$W_1 = \frac{Ml}{2} \text{ and } W_2 = \frac{Ml_2}{2}$$

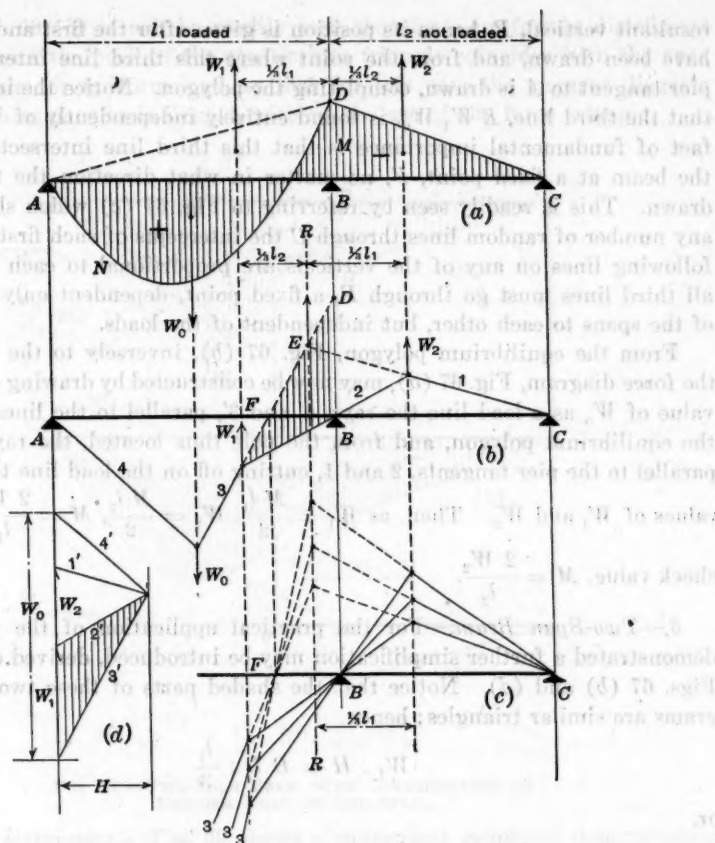


FIG. 67.—TWO-SPAN BEAM, ONE SPAN LOADED.

and, therefore,

$$\frac{W_1}{W_2} = \frac{l_1}{l_2}$$

The lines of action of W_1 and W_2 are verticals, distant $\frac{l_1}{3}$ and $\frac{l_2}{3}$ from the axes of the piers, the "pier verticals", and hence termed the "third verticals". The resultant of W_1 and W_2 is also a vertical, at a distance from W_1 and W_2 inversely proportional to these values, namely, $\frac{l_2}{3}$ from W_1 and $\frac{l_1}{3}$ from W_2 . Hence, it would be suggestive to term this the "resultant vertical" and designate it by R , as in Fig. 67 (b), which shows an equilibrium polygon of W_0 , W_1 , and W_2 . Supposing that the beam under bending is held to the supports, this polygon is obtained, as follows:

The pier tangent to C is drawn at random and from the point where it intersects W_2 the pier tangent to B is drawn intersecting W_1 . The third line of the equilibrium polygon must necessarily intersect the first line on the

resultant vertical, R , hence its position is given after the first and second lines have been drawn, and from the point where this third line intersects W_0 the pier tangent to A is drawn, completing the polygon. Notice the important fact that the third line, $E W_1 W_0$, is found entirely independently of W_0 . Another fact of fundamental importance is that this third line intersects the axis of the beam at a fixed point, F , no matter in what direction the first line was drawn. This is readily seen by referring to Fig. 67 (c) which shows that for any number of random lines through C the intercepts of such first lines and all following lines on any of the verticals are proportional to each other, hence, all third lines must go through F , a fixed point, dependent only on the ratio of the spans to each other, but independent of the loads.

From the equilibrium polygon, Fig. 67 (b), inversely to the general rule, the force diagram, Fig. 67 (d), may now be constructed by drawing on the known value of W_0 as a load line the rays, 4' and 3', parallel to the lines, 4 and 3, of the equilibrium polygon, and from the pole thus located, the rays, 2' and 1', parallel to the pier tangents, 2 and 1, cutting off on the load line the numerical values of W_1 and W_2 . Then, as $W_1 = \frac{M l_1}{2}$, $W_2 = \frac{M l_2}{2}$, $M = \frac{2 W_1}{l_1}$, and the check value, $M = \frac{2 W_2}{l_2}$.

3.—*Two-Span Beam*.—For the practical application of the problem just demonstrated a further simplification may be introduced, derived directly from Figs. 67 (b) and (d). Notice that the shaded parts of these two related diagrams are similar triangles; hence

$$W_1 : H = B D : \frac{l_1}{3}$$

or,

$$\frac{l_1}{2} M : H = B D : \frac{l_1}{3}$$

or, after constructing a force triangle with $\frac{W_0}{l_1}$ as a load line and $\frac{l_1}{3}$ as the pole distance,

$$M : \frac{l_1}{3} = B : \frac{l_1}{3}$$

showing that an equilibrium triangle, $A W_0 D$, derived from this force triangle and drawn though F would cut off the correct value of M on the pier vertical. Fig. 68 (a) shows this construction for a concentrated load on the first span, and Fig. 68 (b) for a uniformly distributed load on the first span. The fixed point, F , is located as before, but by a separate construction, and the equilibrium triangles are drawn this time on $A B$ as a base. Hence, a straight line from A through F (the vertical projection of F on the equilibrium polygon) will cut off the correct value of M on the pier vertical as before, except that it is below B . If the equilibrium line be drawn through F , the desired value of M would be cut off above B as before.

From Fig. 68 (b) it will be readily seen that for a uniformly distributed load on one span the peak of the equilibrium triangle coincides with the apex of the parabola for uniform load, so that for this case the moment diagram may be drawn at once as indicated, without the use of the force triangle.

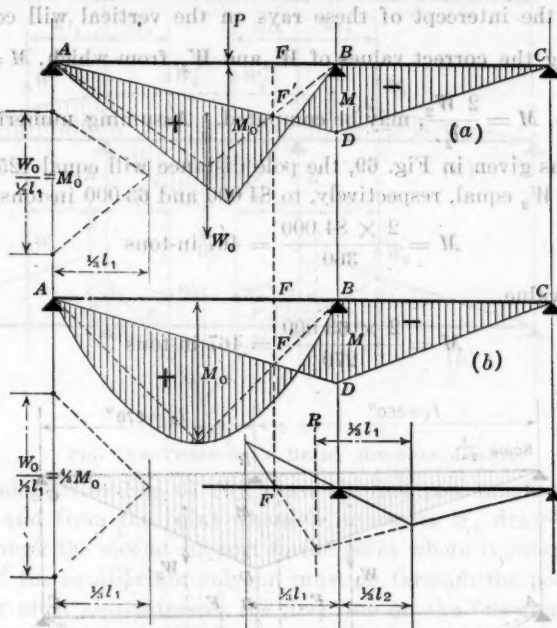


FIG. 68.—TWO-SPAN BEAM WITH CONCENTRATED OR UNIFORM LOAD ON ONE SPAN.

4.—*Pier Depression.*—Fig. 69 shows a convenient graphical determination of the pier moment, M , due to an actual or assumed depression of the mid-support. Consider the beam devoid of all loads including its own weight, but held to the piers, while the support, B , is lowered $\frac{1}{4}$ in. Without knowing the value of M caused by such a depression, the form of the moment diagram is known, for it must be a triangle, $A D C$, as indicated. Each half of the total load triangle is used as an elastic weight for determining the pier tangents, giving,

$$W_1 = \frac{M l_1}{2}; W_2 = \frac{M l_2}{2}; \text{ and } \frac{W_1}{W_2} = \frac{l_1}{l_2}$$

Draw the "third vertical" and the "resultant vertical", R , as before and construct the equilibrium polygon due to W_1 and W_2 . Since the polygon must pass through the contact points of the three supports while its first and third lines must intersect on the resultant vertical, only one such polygon is possible. It may be drawn by trial or constructed by using the fixed point, F , and its projection, F' , of either one or both spans as indicated. Now, if the depression, $B B'$, was plotted to full size ($\frac{1}{4}$ in. in this case) and the spans to a scale of $\frac{1}{240}$, the pole distance of the force diagram to be constructed from the equi-

librium polygon is known; it must equal $\frac{EI}{240}$ in order to correspond to full-sized deflections. Laying off this distance to any convenient scale from a vertical and drawing through the pole thus located three rays parallel to the three pier tangents, the intercept of these rays on the vertical will constitute the load line, giving the correct values of W_1 and W_2 , from which, $M = \frac{2 W_1}{l_1}$, and the check value, $M = \frac{2 W_2}{l_2}$, may be computed. Assuming numerical values of EI , l_1 , and l_2 as given in Fig. 69, the pole distance will equal 125 000 in.-tons, giving W_1 and W_2 equal, respectively, to 84 000 and 63 000 in.-tons, and, hence,

$$M = \frac{2 \times 84\,000}{360} = 467 \text{ in.-tons}$$

and the check value,

$$M = \frac{2 \times 63\,000}{270} = 467 \text{ in.-tons}$$

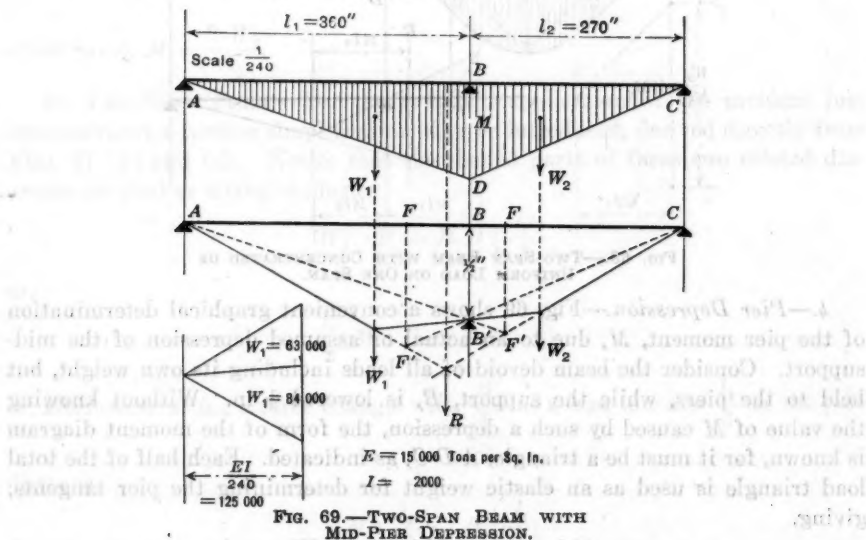


FIG. 69.—TWO-SPAN BEAM WITH MID-PIER DEPRESSION.

The problem may be solved otherwise by considering that the force at B which will deflect the simple beam, AC , $\frac{1}{2}$ in. equals the reaction due to a pier depression of $\frac{1}{2}$ in.

5.—*Three-Span Beam, Mid-Span Loaded.*—Let it be required to determine the pier moments, M' and M'' , Fig. 70 (a), when the mid-span only is loaded.

Without knowing the moments, the form of the moment diagram is known, because it must consist of the "simple moment area" for the mid-span plus a moment triangle for each end span, as shown. In drawing the equilibrium polygon of the pier tangents consider the moment diagram a load area furnishing the five elastic weights, W_0 , W_1 , W_2 , W_3 , and W_4 . The simple moment area, W_0 , is known, or may be computed; it causes downward bending.

The other four elastic weights are unknown because M' and M'' are unknown, but since they are areas of triangles their lines of action must be at the "third verticals" and it is self-evident that they cause upward bending as indicated.

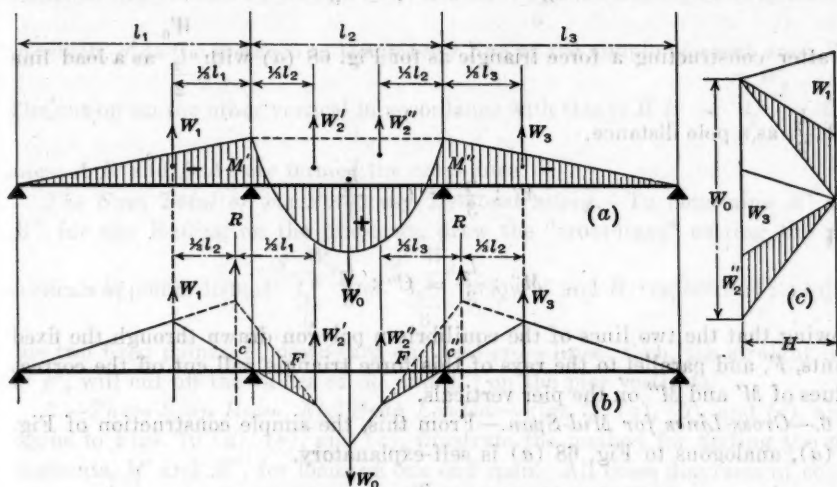


FIG. 70.—THREE-SPAN BEAM, MID-SPAN LOADED.

Now, analogous to Fig. 67 (b), draw the first pier tangent of Fig. 70 (b) at random, and from the point where it intersects W_1 draw the second pier tangent through the second support to the point where it intersects W_2' . The third line of the equilibrium polygon must go through the point just located, and since it must also intersect the first line on the "resultant vertical", R , its position is given regardless of the value, W_0 . A reference to Fig. 67 (c) will show the important characteristic of the third line, namely, that it will pass through a fixed point, F , of the beam axis no matter in what direction the first line was drawn. Now, locate the other fixed point of the span, l_2 , by a separate random construction, as in Fig. 68 (b), and complete Fig. 70 (b). From the equilibrium polygon just drawn a force diagram is constructed, Fig. 70 (c), by drawing six rays parallel to the corresponding six lines of the equilibrium polygon. Since the value, W_0 , is known, the value of the other

four elastic weights are now given to the scale of W_0 , and, because $W_1 = \frac{M' l_1}{2}$ and $W_2' = \frac{M' l_2}{2}$,

$$M' = \frac{2 W_1}{l_1} = \frac{2 W_2'}{l_2}$$

and, similarly,

$$M'' = \frac{2 W_2''}{l_2} = \frac{2 W_3}{l_3}$$

For the practical application of this problem note from the similarity of the shaded triangles of the equilibrium polygon and the force diagram that

$$\frac{l_2}{2} M : H = C : \frac{l_2}{3}$$

and, since the other four elastic weights are unknown because W and M are unknown, but since they are areas of triangles their lines of action must be at the "third points" and it is self-evident that the "third point" bending is indicated.

or, after constructing a force triangle as for Fig. 68 (a) with $\frac{W_0}{2}$ as a load line

and $\frac{l_2}{3}$ as a pole distance,

$$M' : \frac{l_2}{3} = C : \frac{l_2}{3}$$

and

$$M'' : \frac{l_2}{3} = C'' : \frac{l_2}{3}$$

showing that the two lines of the equilibrium polygon drawn through the fixed points, F , and parallel to the rays of this force triangle will cut off the correct values of M' and M'' on the pier verticals.

6.—Cross-Lines for Mid-Span.—From this, the simple construction of Fig. 71 (a), analogous to Fig. 68 (a) is self-explanatory.

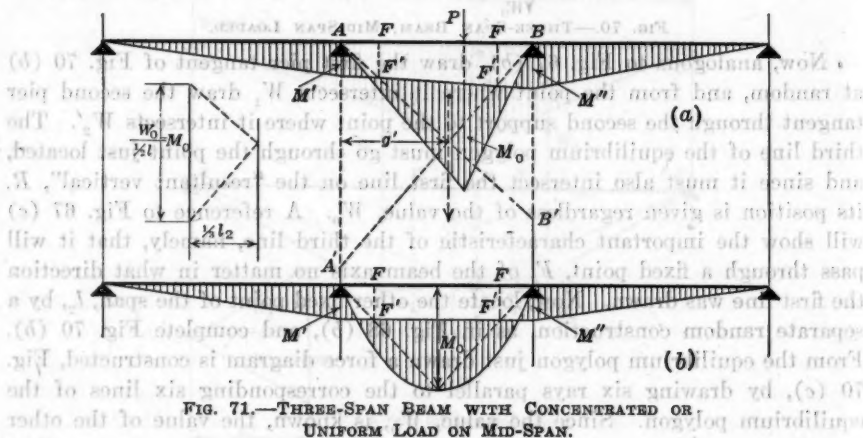


FIG. 71.—THREE-SPAN BEAM WITH CONCENTRATED OR UNIFORM LOAD ON MID-SPAN.

For a uniformly distributed load on mid-span the force triangle is not required, for, analogous to Fig. 68 (b), the two lines of the equilibrium polygon intersect at the apex of the parabola for uniform load, giving Fig. 71 (b).

Indeed, the solution of the problem, namely, to determine M' and M'' , may be made still more general and more simple, giving the same construction for either concentrated or uniformly distributed loading on the mid-span, and requiring no force triangle, by considering the following relation of similar triangles in Fig. 71 (a):

$$\text{Distance, } A A' : g' = \frac{W_0}{2} : \frac{l_2}{3}$$

or, no such third line is now given, for it must intersect the first line on the vertical at A , and the point where this third line intersects W_2 must intersect the resultant vertical of W_1 and W_2 . The fifth and third lines in which, g' is the distance to the gravity line of the simple moment area, W_0 . The cut-off on the other vertical in accordance with this is $B B' = \frac{g'' W_0}{l_2^2}$. The lines, $A B'$ and $B A'$, are termed the cross-lines.

The Sum Total of the Foregoing Demonstrations.—To determine M' and M'' for any loading on the mid-span, draw the "cross-lines" cutting the pier verticals at points distant $\frac{g' W_0}{6}$ and $\frac{g'' W_0}{6}$ below A and B , respectively; project

the two fixed points, F , downward on these cross-lines; then, the straight line, $F' F''$, will cut off the values of M' and M'' on the pier verticals.

7.—*Three-Span Beam, End Span Loaded.*—Figs. 72 (a), (b), and (c), analogous to Figs. 70 (a), (b), and (c), illustrate the method for finding the pier moments, M' and M'' , for loads on one end span. All these diagrams of course are drawn on the assumption that the structure subjected to deformation is held to its supports. Again, without knowing the value of M' and M'' , the form of the moment diagram must be as illustrated in Fig. 72 (a). The reaction at M'' is downward, for the beam would rise from the support if not held to it. The elastic weight, W_0 , the simple beam area, is known, and the unknown elastic weights, W_1 , W_2 , W_2' , and W_3 , since they represent triangular areas, must act at the "third verticals".

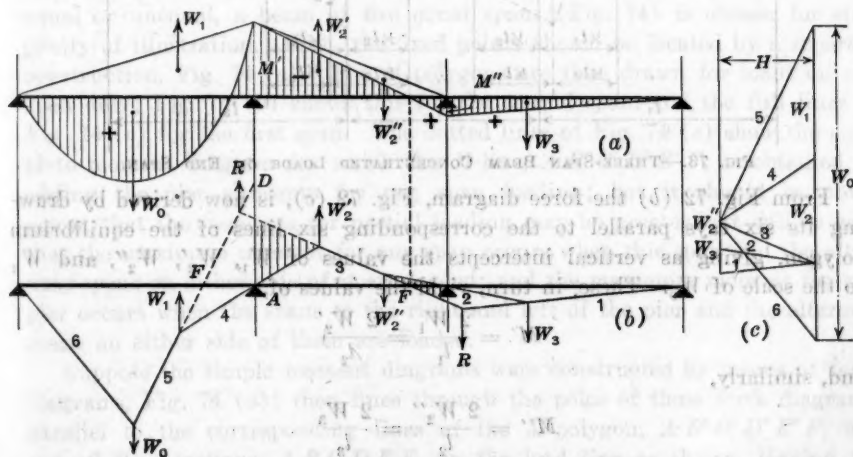


FIG. 72.—THREE-SPAN BEAM, END SPAN LOADED.

In constructing the equilibrium polygon of the pier tangents begin at the right support and draw the first line at random. From the point where it intersects W_3 draw the second line through the support to an intersection with

W_2'' . The third line is now given, for it must intersect the first line on the "resultant vertical", R , and from the point where this third line intersects W_2' another pier tangent is drawn to intersect W_1' . Since the fifth and third lines must intersect the resultant vertical of W_1 and W_2' , the fifth line is now given, and from the point where it intersects W_0 the last pier tangent is drawn, completing the polygon. Referring to Fig. 67 (c) again, the important fact will be evident that the third and fifth lines, termed the "medial lines", in order to distinguish them from the pier tangents, will traverse fixed points, F , for any random first line.

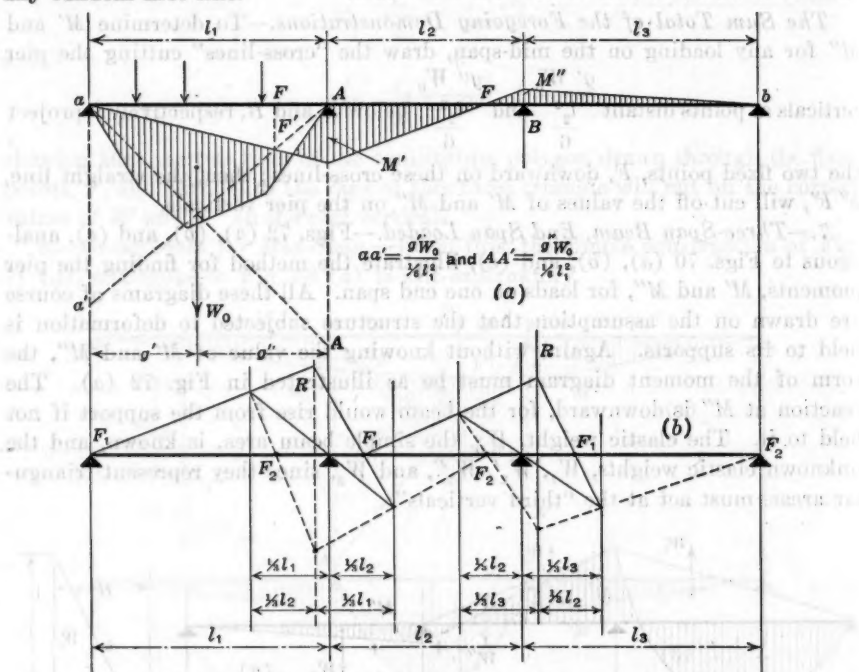


FIG. 73.—THREE-SPAN BEAM CONCENTRATED LOADS ON END SPAN.

From Fig. 72 (b) the force diagram, Fig. 72 (c), is now derived by drawing its six rays parallel to the corresponding six lines of the equilibrium polygon, giving as vertical intercepts the values of W_1 , W_2' , W_2'' , and W_3 to the scale of W_0 . These, in turn, yield the values of,

$$M' = \frac{2 W_1}{l_1} = \frac{2 W_2'}{l_2}$$

and, similarly,

$$M'' = \frac{2 W_2''}{l_2} = \frac{2 W_3}{l_3}$$

8.—Cross-Lines for End Span.—For the end span loaded, reasoning identically the same as for the mid-span loaded, leads to constructions like Figs. 71 (a) and (b) and, finally, to the more general method of cross-lines, Fig. 73 (a).

It has been shown that the cut-off by a medial line of the loaded span on the pier vertical has a definite relation to the pier moments. It is further

important to remember that the cut-offs by the medial lines of any unloaded span on the pier verticals are proportional to the pier moments. This latter fact is apparent when noting that the two shaded triangles of Fig. 72 (b) are similar to the corresponding two triangles on $W_2' = \frac{M' l_2}{2}$ and on $W_2'' = \frac{M'' l_2}{2}$,

Fig. 72 (c); hence, it follows that the moment line of an unloaded span, like the medial line of such a span, must traverse the fixed point, F . Since the pier moments decrease from the loaded span toward each end of the structure, the point, F , through which the moment line passes is necessarily always the second one in that direction. Hence, referring again to Fig. 73 (a), it is only necessary to determine M' , for the moment line from that point to the end must pass through the fixed points, F . The end support is the last fixed point.

It is self-evident that this reasoning applies to continuous beams of any number of supports. If the loaded span is not an end span it is only necessary to determine the two pier moments to the right and left of the loaded span by "cross-lines" and complete the moment diagram by drawing the remaining moment lines through the fixed point, F . A glance at Fig. 67 (b) and Fig. 68 (b), and an examination of Fig. 72 (b) will show that by a random construction like Fig. 73 (b), the fixed points, F , may be located for any number of spans. In Fig. 73 (b), proceeding from left to right, the full lines locate the points, F_1 , and, from the other direction, the dotted lines locate the points, F_2 . For equal spans, the resultant verticals coincide, of course, with the pier verticals.

9.—*Continuous Beams of Any Number of Spans.*—To recapitulate and show the general procedure for continuous beams of any number of spans, equal or unequal, a beam of five equal spans (Fig. 74) is chosen for simplicity of illustration. First, the fixed points should be located by a separate construction, Fig. 74 (a). The M -polygons are then drawn for loads on one span only; Fig. 74 (b) shows this for the second span, and the full lines of Fig. 74 (c) for the first span. The dotted lines of Fig. 74 (c) show the complete moment diagram for which the M -line, $A B' C' D' E' F$, is obtained by adding the pier moments for one span loading; but it should be noted again, that the moments for partial loading may be greater. It will be seen that the maximum moment for any span occurs when this span and the alternate spans on either side of it are loaded; and the maximum moment for any pier occurs when the spans to the right and left of the pier and the alternate spans on either side of them are loaded.

Suppose the simple moment diagrams were constructed by means of force diagrams, Fig. 74 (d); then lines through the poles of these force diagrams, parallel to the corresponding lines of the M -polygon, $A B' C' D' E' F$, will cut off the reactions, $A B C D E F$, on the load line as shown. Having the reactions, the shear diagram, Fig. 74 (e), can be obtained. If it is desired to refer all moments to a horizontal base line, Fig. 74 (f), the diagram may be constructed by using the poles, O' , of Fig. 74 (d). This would afford an acute check on the accuracy of the whole work.

It will be seen from Figs. 74 (b) and (c) that the moments for one span loaded diminish rapidly toward the ends, and it seems sufficient in practice to extend the investigation for maximum moments over a radius of two adjacent spans only.

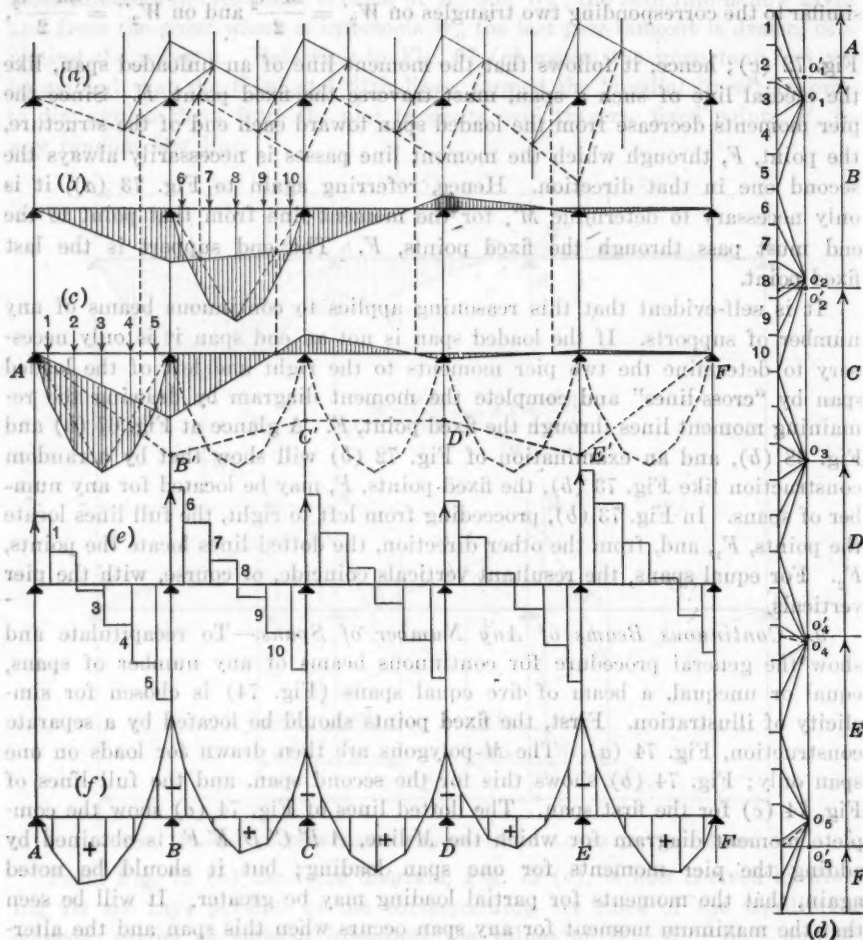


FIG. 74.—TYPICAL CONSTRUCTIONS FOR CONTINUOUS BEAMS OF ANY NUMBER OF SPANS.

10.—*One-Span Beam, Fixed Ends.*—The methods demonstrated may be applied of course to the most simple case, the one span beam fixed at both ends. To obtain the moments, draw the equilibrium polygon of end tangents (Fig. 75 (a)), and from it the force diagram, which, in turn, will yield the moments, M_1 and M_2 , analogous to Fig. 67 (b). Since the "medial lines" must pass through the fixed points, these latter are seen to be distant $\frac{l}{3}$ from the ends, and may be used in constructing the moment diagram by "cross-lines", Fig. 75 (b), analogous to Figs. 71 (a) and 73 (a).

Diagrams like this are well adapted for deriving formulas for typical loading. For the simple case of a uniformly distributed load, p per ft., no construction is necessary. The simple moment is $\frac{p l^2}{8}$, hence, the parabolic area,

$$W_0 = \frac{p l^2}{8} \times \frac{2 l}{3} = \frac{p l^3}{12}$$

and since $W_1 = W_2 = \frac{1}{2} W_0$ in this case,

$$M_1 = M_2 = \frac{\frac{1}{2} W_0}{\frac{l}{2}} = \frac{p l^2}{12}$$

leaving a moment for mid-span of $\frac{p l^2}{24}$ as shown in Fig. 75 (c).

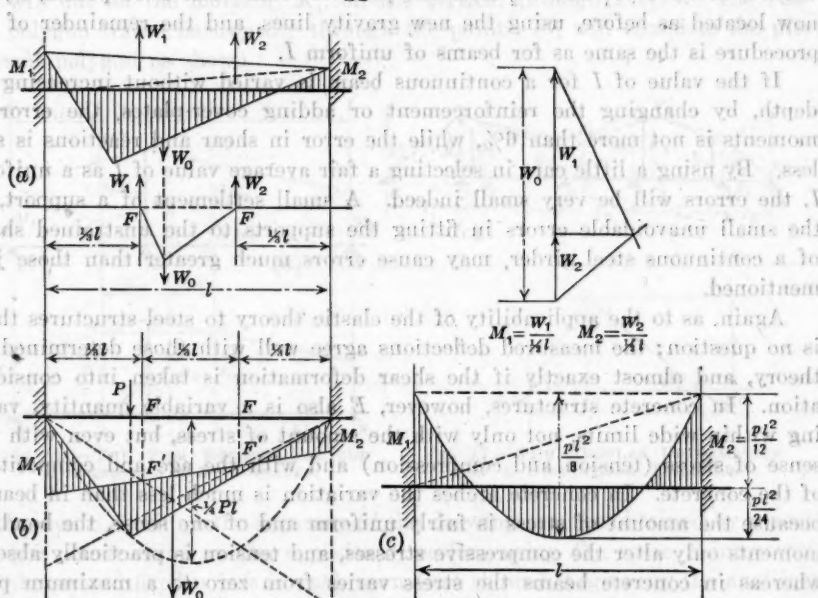


FIG. 75.—BEAMS FIXED BOTH ENDS.

11.—*Consideration of Variable Moment of Inertia.*—If I varies from span to span only, being uniform throughout each span, the constructions remain the same except for the location of the resultant verticals, the "T-lines" of the authors, as explained by Fig. 15*, or by transformed spans as will be treated in Fig. 78 (b).

The procedure for the case of variable moments of inertia throughout, although not differing from the foregoing demonstrations in theory, becomes more involved in practical application. The elastic weights, $\frac{M}{E I}$, reduce to

M for a constant, $E I$, and to $\frac{M}{I}$ for a constant, E , only; but in order to avoid small fractional figures, and to make all reduction factors nearly equal to unity for convenience in application, the elastic weights are made $\frac{M C}{I}$, C being a constant equal to the greatest, the smallest, or any average I occurring most frequently. Hence, all ordinates of the simple moment areas as well as of the triangular M -areas must be multiplied by $\frac{C}{I}$ in order to obtain the "modified" elastic load areas. Then the gravity lines of these new areas must be determined, being the lines of action of the new elastic weights, W_0, W_1, W_2 , etc. Note that the third lines, being no longer gravity lines of triangles, have shifted slightly, but the resultant vertical is located as before, that is, by transposing the distances of W_1 and W_2 from the pier verticals. The fixed points, F , are now located as before, using the new gravity lines, and the remainder of the procedure is the same as for beams of uniform I .

If the value of I for a continuous beam is varied without increasing its depth, by changing the reinforcement or adding cover-plates, the error in moments is not more than 6%, while the error in shear and reactions is still less. By using a little care in selecting a fair average value of I as a uniform I , the errors will be very small indeed. A small settlement of a support, or the small unavoidable errors in fitting the supports to the unstrained shape of a continuous steel girder, may cause errors much greater than those just mentioned.

Again, as to the applicability of the elastic theory to steel structures there is no question; the measured deflections agree well with those determined by theory, and almost exactly if the shear deformation is taken into consideration. In concrete structures, however, E also is a variable quantity, varying within wide limits, not only with the amount of stress, but even with the sense of stress (tension and compression) and with the age and composition of the concrete. In concrete arches the variation is much less than in beams, because the amount of stress is fairly uniform and of one sense, the bending moments only alter the compressive stresses, and tension is practically absent, whereas in concrete beams the stress varies from zero to a maximum plus or minus. In trying to find an answer to the question of "what is the E and I of a reinforced concrete beam that will yield a deflection within reasonable agreement with the measured deflection", a great refinement in the analysis of reinforced concrete beams and slabs must seem illusory.

12.—*Another Method of Constructing the M-Polygon.*—A practical and useful line relation is obtained as follows: The intercepts of the moment diagram on the "third verticals", Fig. 76 (a), are:

$$T_1'' = \frac{1}{3} M_1 + \frac{2}{3} M_2 \text{ and } T_2' = \frac{2}{3} M_2 + \frac{1}{3} M_3$$

Hence,

$$T_1'' l_1 + T_2' l_2 = \frac{1}{3} [M_1 l_1 + 2 M_2 (l_1 + l_2) + M_3 l_2] = \frac{1}{3} N_2$$

equals one-third the load term, N_2 , because the expression in the parentheses is the "three-moment equation". Drawing now the line, mn (Fig. 76 (b)), the intercept on the "resultant vertical" is,

$$R_2 = \frac{T_1'' l_1 + T_2' l_2}{l_1 + l_2} = \frac{N_2}{3(l_1 + l_2)} \dots \dots \dots (92)$$

By means of these R -intercepts on the resultant verticals, together with the fixed points, F , the pier moment polygon may be drawn for any loading whatever, as illustrated by Fig. 76 (c). First plot the values, R_1 , R_2 , R_3 , and R_4 , on the resultant verticals as indicated by the heavy ordinates. Then draw the auxiliary polygon, $F_0, F_1', F_2', F_3', F_4',$ and F_5 , each line through the top of each R -ordinate until it intersects the vertical through the fixed point, F . The intersections, $F_1', F_2',$ etc., are points of the required pier moment polygon, and the last line, $F_4' F_5$, is at once a line of such polygon and will cut off the moment, M_4 , on the vertical through Pier 4. The full-line polygon traced backwardly through the points, F' , will complete the pier moment polygon as shown.

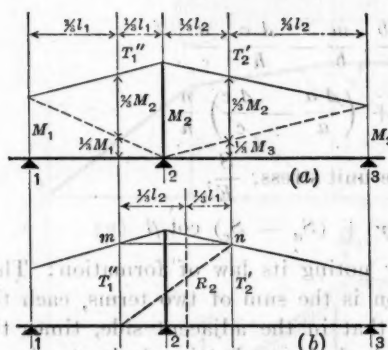


FIG. 76.—THE PIER MOMENT POLYGON DETERMINED BY R -ORDINATES.

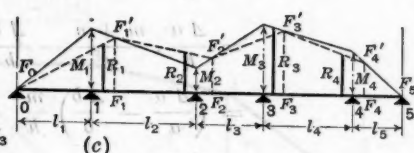


FIG. 77.—THE ANGULAR DEFORMATION OF A STRUCTURAL TRIANGLE.

An acute check on the accuracy of the drawing may be obtained by repeating the construction in the opposite direction, using the same R -intercepts, but the second fixed points in each span (not shown), beginning the auxiliary polygon at F_5 .

For a uniformly distributed load, p , per linear foot, the simple moment area = $\frac{1}{8} p l^2$. $\frac{2}{3} l = \frac{1}{12} p l^3$, and its reaction at the pier = $\frac{1}{24} p l^3$. Hence, the load term of the three-moment equation for Pier 2, since both sides of the equation were multiplied by $6 EI$ (see the authors' derivation*) is,

$$N_2 = \frac{1}{4} p (l_1^3 + l_2^3)$$

which gives an R -intercept of,

$$R_2 = \frac{p}{12} \left(\frac{l_1^3 + l_2^3}{l_1 + l_2} \right)$$

* Proceedings, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1595.

For a pier depression, d_2 , the pier reaction is,

$$\frac{d_2}{l_1} + \frac{d_2}{l_2} = d_2 \frac{l_1 + l_2}{l_1 l_2}$$

Hence, as before,

$$N_2 = 6 E I \frac{d_2 (l_1 + l_2)}{l_1 l_2}$$

and,

$$R_2 = \frac{2 E I d_2}{l_1 l_2}$$

13.—*Application to Secondary Stresses.*—The basis of the subject of secondary stress is the deformation of a structural triangle. If the three sides of a triangle, Fig. 77, change their lengths, a , b , and c , by the amounts, Δa , Δb , and Δc , respectively, the change of the angle, α , is

$$\begin{aligned} \Delta \alpha &= \frac{\Delta a}{h} - \frac{\Delta b \cos \gamma}{h} - \frac{\Delta c \cos \beta}{h} \\ &= \frac{\Delta a}{h} \cdot \frac{m+n}{a} - \frac{\Delta b}{h} \cdot \frac{m}{b} - \frac{\Delta c}{h} \cdot \frac{n}{c} \\ &= \left(\frac{\Delta a}{a} - \frac{\Delta b}{b} \right) \frac{m}{h} + \left(\frac{\Delta a}{a} - \frac{\Delta c}{c} \right) \frac{n}{h} \end{aligned}$$

Since the unit strain, $\frac{\Delta a}{a}$, etc., equals the unit stress, $\frac{S}{E}$,

$$E \Delta \alpha = (S_a - S_b) \cot \gamma + (S_a - S_c) \cot \beta$$

This equation is readily remembered by noting its law of formation: That for each angle of a triangle the equation is the sum of two terms, each the unit stress in the opposite side minus that in the adjacent side, times the cotangent of the included angle. The check for each triangle is $\Delta \alpha + \Delta \beta + \Delta \gamma = 0$.

Let Fig. 78 (a) represent a truss having a top chord of continuous construction between the pins, 0 and 5. Considering chord deformation only, the problem is simplified by regarding the top chord as a continuous beam on the panel points as supports. The moment reaction for any panel point (its elastic weight) is equal to its angle of deformation; hence, for Panel Point 2:

$$\frac{1}{3} M_1 \pm \frac{2}{3} M_2 \cdot \frac{l_2}{2} + \frac{2}{3} M_2 \pm \frac{1}{3} M_3 \cdot \frac{l_3}{2} = \Delta \phi_2$$

or,

$$M_1 \frac{l_2}{I_2} + 2 M_2 \left(\frac{l_2}{I_2} + \frac{l_3}{I_3} \right) + M_3 \frac{l_3}{I_3} = 6 E \Delta \phi_2$$

Multiplying this equation by a constant, I_0 , in order to obtain convenient numerical values, and introducing the abbreviation,

$$l_2 \frac{I_0}{I_2} = l_2' \text{ and } l_3 \frac{I_0}{I_3} = l_3'$$

gives, making the depth of any member as small as practicable, consistent with the

$$M_1 l_2' + 2M_2 (l_2' + l_3') + M_3 l_3' = 6 E I_c \Delta \phi_2$$

which again is seen to be the three-moment equation, in this case with the load term, $N_2 = 6 E I_c \Delta \phi_2$. Hence by Equation (92),

$$R_2 = \frac{2 E I_c \Delta \phi_2}{l_2' + l_3'} \quad (93)$$

Fig. 78 (b) shows the construction for the upper chord, 0-5, of Fig. 78 (a) and is typical for all such cases where the two end moments are zero. For a symmetrical structure symmetrically loaded the M -polygon will appear in symmetrical form and thus check itself. For any unsymmetrical conditions a check construction may be had by using the same R -intercepts but the second fixed point in each span, and beginning the auxiliary F' -polygon at Point 5, exactly as explained for Fig. 76 (c).

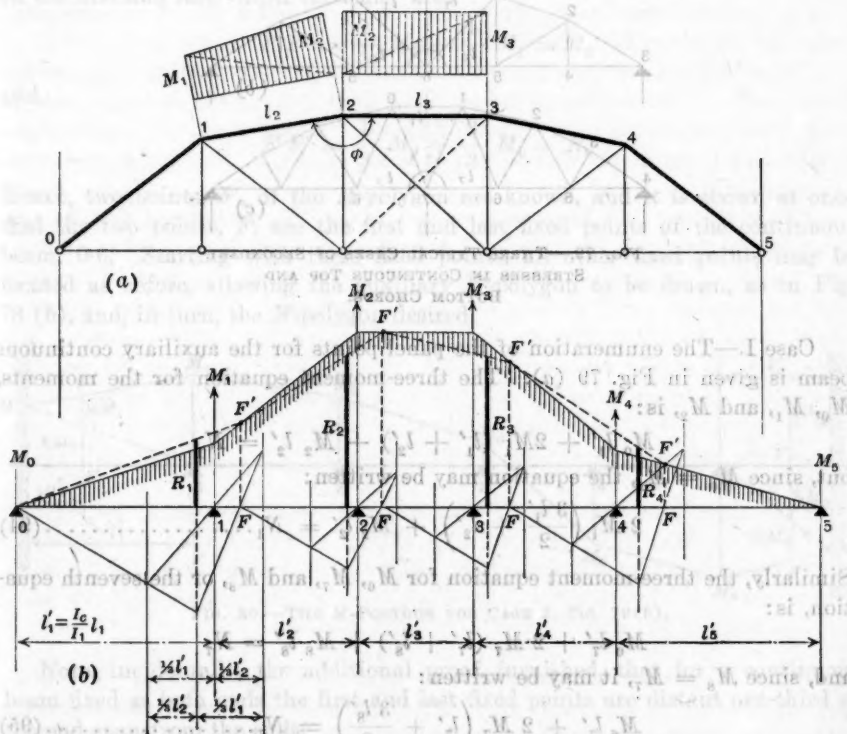


FIG. 78.—SECONDARY STRESSES IN A CONTINUOUS TOP CHORD.

Note, as appears at a glance from Fig. 78 (b), that the intercepts, R , alone give a fair approximation of the moments, M_1, M_2, M_3 , etc., entirely sufficient for a practical estimate of the secondary stresses to be expected. Further, since l_n' is inversely proportional to I_n , the greater the moment of inertia, I , the greater the values of R and hence of M , pointing to the advisability of

making the depth of any member as small as practicable, consistent with other considerations.

14.—*Circumferential Chords of Trusses.*—If for a circumferential upper and lower chord no moment is zero but the structure is symmetrical, the determination of the panel-point moments does not differ greatly from the problem just demonstrated. The following three cases will cover such problems:

I.—The axis of symmetry contains no panel point, Fig. 79 (a);

II.—The axis of symmetry contains two panel points, Fig. 79 (b); and,

III.—The axis of symmetry contains one panel point, Fig. 79 (c).

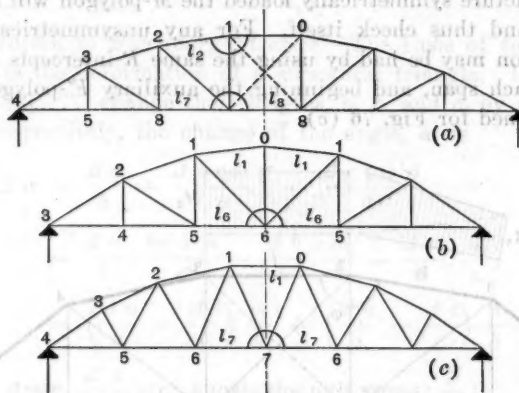


FIG. 79.—THREE TYPICAL CASES OF SECONDARY STRESSES IN CONTINUOUS TOP AND BOTTOM CHORDS.

Case I.—The enumeration of the panel points for the auxiliary continuous beam is given in Fig. 79 (a). The three-moment equation for the moments, M_0 , M_1 , and M_2 , is:

$$M_0 l'_1 + 2M_1 (l'_1 + l'_2) + M_2 l'_2 = N_1$$

but, since $M_0 = M_1$, the equation may be written:

$$2 M_1 \left(\frac{3 l'_1}{2} + l'_2 \right) + M_2 l'_2 = N_1 \dots \dots \dots (94)$$

Similarly, the three-moment equation for M_6 , M_7 , and M_8 , or the seventh equation, is:

$$M_6 l'_7 + 2 M_7 (l'_7 + l'_8) + M_8 l'_8 = N_7$$

and, since $M_6 = M_7$, it may be written:

$$M_6 l'_7 + 2 M_7 \left(l'_7 + \frac{3 l'_8}{2} \right) = N_7 \dots \dots \dots (95)$$

Equations (94) and (95) show that the auxiliary continuous beam, 0-8, has freely supported ends, because the two end terms, $M_0 l'_1$ and $M_8 l'_8$, respectively, have vanished from these two equations. Hence, the M -polygon will be analogous to Fig. 78 (b) except that the first and last spans are, respectively:

$\frac{3 l'_1}{2} = \frac{3 l_1}{2} \frac{I_c}{I_1}$ and $\frac{3 l'_8}{2} = \frac{3 l_8}{2} \frac{I_c}{I_8}$

from which,

$$R_1 = \frac{2 E I_c \Delta \phi_1}{\frac{3 l_1'}{2} + l_2'}, \text{ and } R_7 = \frac{2 E I_c \Delta \phi_7}{l_7' + \frac{3 l_8'}{2}}$$

Case II.—The three-moment equation for Point 0 is:

$$M_1 l_1' + 2M_0 (l_1' + l_1') + M_1 l_1' = N_0$$

divided by 3 it gives,

$$\frac{2}{3} M_0 + \frac{1}{3} M_1 = \frac{N_0}{3 (l_1' + l_1')} = R_0$$

and, similarly,

$$\frac{1}{3} M_5 + \frac{2}{3} M_6 = \frac{N_6}{3 (l_6' + l_6')} = R_6$$

If now Fig. 80 represents the moment polygon desired, its intercepts, FF' , on the first and last "third verticals" are:

$$FF' = \frac{2}{3} M_0 + \frac{1}{3} M_1 = R_0$$

and,

$$FF' = \frac{1}{3} M_5 + \frac{2}{3} M_6 = R_6$$

Hence, two points, F' , of the M -polygon are known, and it is shown at once that the two points, F , are the first and last fixed points of the continuous beam, 0-6. Starting with these fixed points, all other fixed points may be located as before, allowing the auxiliary F' -polygon to be drawn, as in Fig. 78 (b), and, in turn, the M -polygon desired.

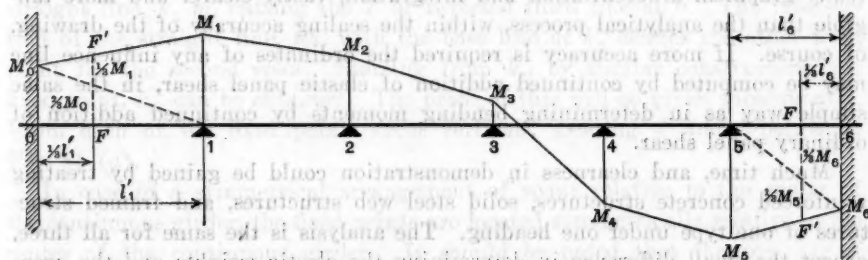


FIG. 80.—THE M -POLYGON FOR CASE 2, FIG. 79(b).

Note, incidentally, the additional proof furnished, that for a continuous beam fixed at both ends the first and last fixed points are distant one-third of the end span from the ends.

Case III.—This is evidently a combination of Cases I and II. The auxiliary continuous beam is freely supported at Point 0, corresponding to Fig. 78 (b), while the other end at Point 7 is fixed, corresponding to Fig. 80. Hence, for the first span,

$$l_1' = \frac{3 l_1}{2} \frac{I_c}{I_1}$$

while for all the other spans,

$$r = l \frac{I_c}{I}$$

The second fixed point of the end span is $\frac{l'}{3}$ from the end; and its ordinate is,

$$R_2 = \frac{2 E I_c \Delta \phi_2}{l' + l'}$$

For a complete determination of secondary stresses, including web members, methods similar to the one just demonstrated could be used by considering the three members of each structural triangle a continuous beam on four supports, the first coinciding with the last, but such methods become cumbersome, and the computation of secondary stresses by the method of normal equations* is superior for such cases.

General.—In conclusion, the writer wishes to add a few words regarding statically indeterminate problems in general.

The whole subject can be demonstrated, in an elementary way, within wide limits of practical application, with little tax on the memory, that is, with few elements as a base, namely, similar triangles, force diagram, and equilibrium polygon, the elastic weight, and Maxwell's "theorem of reciprocal deformation".

This statement will seem less radical when it is considered that in constructing an influence line (a line of deflection) one has determined the numerical values of strings of differentials; in determining the area of a graph one has similarly performed numerical integration. The whole process is really graphical differentiation and integration, vastly clearer and more tangible than the analytical process, within the scaling accuracy of the drawing, of course. If more accuracy is required the ordinates of any influence line may be computed by continued addition of elastic panel shear, in the same simple way as in determining bending moments by continued addition of ordinary panel shear.

Much time, and clearness in demonstration could be gained by treating reinforced concrete structures, solid steel web structures, and framed structures of one type under one heading. The analysis is the same for all three, except the small difference in determining the elastic weights and the accuracy desired.

R. McC. BEANFIELD,† Assoc. M. Am. Soc. C. E. (by letter).‡—The practical application of graphical analysis to the solution of problems involving continuous beams and indeterminate frames is now rather conspicuous by reason of its limited use by the profession in general. It is the exception rather than the rule for the average designing engineer to make a complete and proper stress analysis of statically indeterminate problems, particularly

* "Secondary Stresses in Bridges," by Cecil Vivian von Abo, Jun. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, p. 969.

† Structural and Mech. Engr., Los Angeles, Calif.

‡ Received by the Secretary, December 24, 1925.

in reinforced concrete construction, due to the laborious, slow, and cumbersome analytical methods, and to the difficulties of obtaining a proper check. In general, if engineers had a more practical working knowledge of graphical analysis applied to statically indeterminate structures, as suggested by the authors, they would have confidence in their work resulting in safer and more economical structures.

The writer has been using, for a number of years, a system of graphical analysis which is an adaptation, in general, of Professor W. Ritter's method,* and is somewhat simpler than that of the authors. Professor Ritter's graphical analysis seems to be little known or understood by American engineers, possibly due to the fact that a proper translation into English has never been published. Prior to discussing the advantages of Ritter's graphical method, a complete graphical solution by that method, using the authors' example and their notations wherever possible, will be given.

Construct the R , U , V , T -lines and the simple-beam moment diagrams for the various spans as described by the authors. (Fig. 81 (b) and (a), respectively.) In Fig. 81 (b) starting from J_1 draw any straight line that will cut the V or trisecant line and the first T or reversed third line in the points, e and f , respectively. Draw the straight line, ef , which, prolonged, will cut the U -line in the second span in the point, g . Join g and f by a straight line intersecting the base line in the point, J_2 , called a fixed point in the second span.

Starting from the fixed point, J_2 , repeat the same process, thus obtaining the second fixed point, J_3 , in the third span. The fixed points, k_1 and k_2 , are found in a similar manner starting from k_3 , Fig. 81 (b). Draw any straight line, $k_3 m$, cutting the T or reversed third line at n . The intersection of the straight line, nh , with the base line at k_2 defines the other fixed point for the second span. Starting from k_2 , by similar construction, the fixed point, k_1 , can be obtained. The proof of this will be given subsequently. From each of the fixed points erect verticals, locating J and k -points in Fig. 81 (a).

In case of a symmetrical arrangement of spans relative to the center of the continuous girder, the fixed points are located symmetrically relative to the center of the continuous girder. It should be noted that the position of the fixed points is dependent only on the relation of the spans to each other and not in any way on the loading of the spans. After the fixed points are located, the bending moment of each load on all supports is determined, considering only one span to be loaded at a time, usually the left-hand span. The algebraic summation of the moments (intercepts) at the supports due to the separate loads will give the resultant moments due to the entire system of loads on the continuous girder. The final closing line is drawn through these points, or final intercepts at the supports.

The bending moments for a uniformly loaded span in a continuous girder are found graphically, as follows:

* "Anwendungen der Graphischen Statik," by C. Culmann; "Der Kontinuierliche Balken," by W. Ritter, 1900.

Construct the simple-beam moment diagram (parabola, Fig. 82 (a)). Connect the point, S , at the apex of the parabola with the points of supports of the loaded span. The intersection of these lines with the fixed lines (i and k -lines) give the points, i_1' and k_2' , which are points in the closing line of the moment area of the girder. (See Fig. 82 (a) and (b).) When more than one span is loaded, treat each span separately and summarize the results.

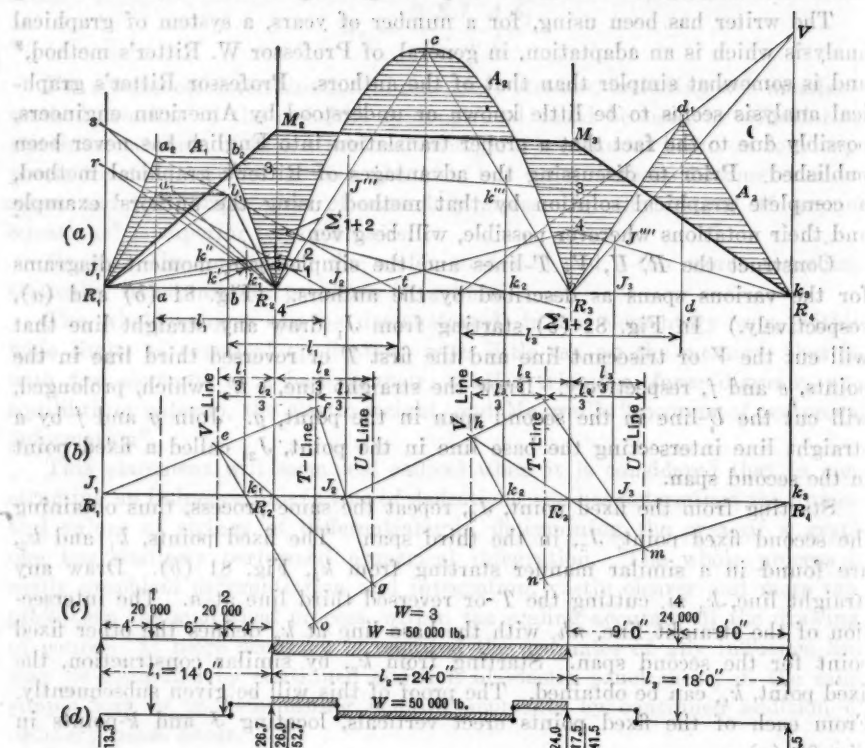


FIG. 81.

For a single concentration or moving load the moments or their intercepts on the reaction verticals are found, as follows: Refer to Fig. 83 (the proof will be given subsequently). Lay off on the base line the length, l_2 , on each side of the load. From the points thus found draw lines through the vertex, S , of the simple-beam moment area, bSc . These lines intersect the support perpendiculars in b'' and c'' . From the points, b'' and c'' , draw lines to c and b , respectively. The respective intersections of cb'' and bc'' with verticals through the fixed points, i_2 and k_2 , are points on the closing lines, which, extended, intersect the verticals through the supports at b_1 and c_1 . The intercepts, bb_1 and cc_1 , are the moments measured to the scale of moments, at the supports. The moment closing line for the load, P , is extended from b_1 to a and from c_1 through the fixed point, k , to d , and continued to e . The moment closing line indicates the moment effects on the other spans for the concentrated load, P , on the second span.

The zero moment or inflection point, i , lies between the support and the fixed perpendicular, $i_2 i_1'$, and the inflection point, k , between the fixed line, $k_2 k_2'$, and the support. From this it follows that in a middle span as a single load moves from left to right the inflection points, i and k , in the bending diagram move in the same direction, but always remain inside the loaded span, l_2 , and outside the fixed distance, $i_2 k_2$.

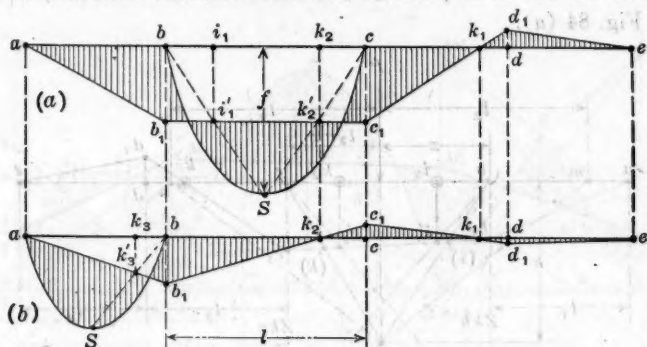


FIG. 82.

To determine the moments due to the combined effects of the various loads refer to Fig. 81 (a) (the authors' example) which is a continuous girder with freely supported ends. From similar construction to that shown in Fig. 83, for a single concentrated load draw the closing lines, $R_1 k'1$, and $R_1 k''2$, which intersect the vertical through the support, R_2 , with segments or intercepts, R_2-1 and R_2-2 , respectively, measured to the scale of moments. These intercepts measure the moments at the supports due to Loads 1 and 2, respectively. The summation of these intercepts or segments, equals the segment, $R_2-\Sigma 1+2$. Through $\Sigma 1+2$ on R_2 produce a line passing through the fixed point, k_2 , and intersecting R_3 at $\Sigma 1+2$. The segment, $R_3-\Sigma 1+2$, equals the moment, measured to scale of moments, due to Loads 1 and 2 at R_3 .

The closing line for the uniform load on the second span is 3, $j''' k''' 3$. The segments, R_2-3 and R_3-3 , are the moments at these supports induced by the uniform load, 3, on the second span.

Similarly, the concentrated load, 4, produces moments at R_3 equal to the segment, R_3-4 , and at R_2 equal to the segment, R_2-4 , respectively. All segments above the base line, $R_1 R_4$, are positive and those below are negative. An algebraic summation of the segments or intercepts at each support line, such as R_2-M_2 and R_3-M_3 , give the M -points through which the final closing line, $R_1 M_2 M_3 R_4$, can be drawn. The writer suggests that the summation of the moment intercepts on the support or R -lines can be easily and accurately accomplished by a good pair of dividers.

The difference between the ordinates of the simple-beam moment diagrams and those of the diagram, $R_1 M_2 M_3 R_4$, will give the resultant moment diagram shown shaded in Fig. 81 (a). All areas above the closing line, $R_1 M_2 M_3 R_4$, give positive moments and those below negative moments. The intersection of the final closing line with the simple beam curves gives the points of inflection.

Fig. 84 (a) shows the solution of the authors' example assuming the ends of the continuous girder to be fixed. The graphical construction is identical with that of Fig. 81 (a) and (b) except that the initial fixed points are located on the first and last trisecant V or U -lines instead of on the extreme ends of the girder. The moment closing lines are located by passing through the proper intersections on the fixed point or extreme trisecant verticals, J_1 or k_3 , as shown in Fig. 84 (a).

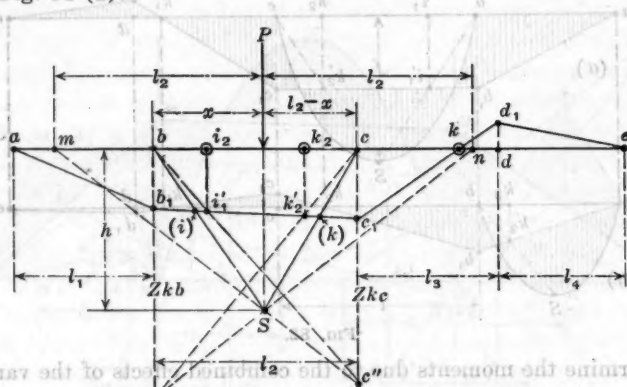


FIG. 83.

In a continuous girder with partly restrained end conditions the fixed point is located between R_1 and $\frac{L}{3}$ in proportion to the relative degree of fixity considered. The moment closing lines are then drawn passing through the proper intersections on the fixed points, otherwise the procedure is similar to that shown in Fig. 81 (a).

The advantages of Professor Ritter's method when compared to the authors' system using conjugate or characteristic points are as follows:

(a).—Elimination of the computation for moment areas and of dividing them by the length of span; also of the computations of the centers of gravity of the moment areas, which for some conditions of loading become rather complicated and laborious. The elimination of computations lessens the possibilities of errors.

(b).—In the method advocated by the writer the influence of each load relative to the effects of moments on the entire continuous girder can be readily ascertained and visualized without additional operations or separate diagrams. This advantage is particularly valuable when investigating a continuous girder for live loads on alternate spans or for unbalanced loading, also for moving loads and the construction of influence lines for them. For the general solution and the construction of the influence lines the methods of Professor Ritter* are doubtless much simpler and shorter than those advocated by the authors.

* "Taschenbuch für Bauingenieur," E. H. Max Foerster, Fourth Edition, pp. 361-363.

(c).—Practical examples of the authors' graphical system involving the use of conjugate points discloses the fact that for certain conditions of loading and for varying spans the "pennant diagrams" become so flattened and the angles so acute that the intersections locating the conjugate points are very difficult to ascertain.

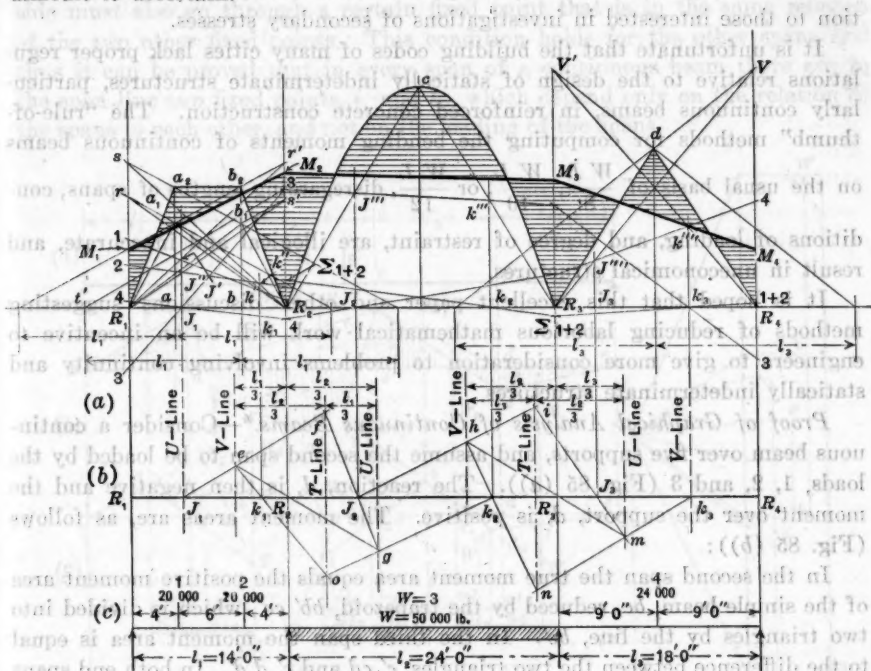


FIG. 84.

For the determination of shears and reactions, the following method will be found advantageous. (Fig. 81 (c) and (d)). The continuous girder may be considered as a series of cantilevers over the supports, with simple suspended spans hung at the points of contraflexure. The total reaction of any support would then be the total load on the cantilever plus the reactions from the adjacent suspended spans. The point of zero shear is located by constructing an ordinate through the point of maximum positive bending moment.

In the design of continuous beams proper consideration should be given to the maximum positive moments due to partial or unbalanced loading of the spans. For ordinary conditions the writer suggests the following rule: Design for full moments at the supports and for four-thirds of the positive moments between the supports for all spans fully loaded. Where unusual conditions of loading occur, the positive moment between the supports must be carefully investigated as it may exceed four-thirds of the positive moment for all spans fully loaded. The graphical analysis suggested by the writer provides for quick and easy investigation for partial loading for the moment effects on all spans.

The suggestions given in the paper for solving problems involving members with varying moments of inertia and statically indeterminate frames can be applied in like manner to the graphical analysis advocated by the writer. The author's graphical construction as an aid in the solution of secondary stress problems is rather ingenious and worthy of much consideration to those interested in investigations of secondary stresses.

It is unfortunate that the building codes of many cities lack proper regulations relative to the design of statically indeterminate structures, particularly continuous beams, in reinforced concrete construction. The "rule-of-thumb" methods for computing the bending moments of continuous beams on the usual basis of $\frac{WL}{8}$, $\frac{WL}{10}$, or $\frac{WL}{12}$, disregarding lengths of spans, conditions of loading, and degree of restraint, are illogical and inaccurate, and result in uneconomical structures.

It is hoped that this excellent paper and other discussions suggesting methods of reducing laborious mathematical work will be an incentive to engineers to give more consideration to problems involving continuity and statically indeterminate structures.

*Proof of Graphical Analysis of Continuous Beams.**—Consider a continuous beam over five supports, and assume the second span to be loaded by the loads, 1, 2, and 3 (Fig. 85 (a)). The reaction, d , is then negative and the moment over the support, d , is positive. The moment areas are, as follows (Fig. 85 (b)):

In the second span the true moment area equals the positive moment area of the simple beam, bc , reduced by the trapezoid, $bb'cc'$, which is divided into two triangles by the line, bc' . In the third span the moment area is equal to the difference between the two triangles, $c'cd$ and $c'd\bar{c}$. In both end spans the moment area is always a triangle (for loads on the interior spans only).

1.—In the center of gravity of the triangles and of the simple moment area of the loaded span, Fig. 85 (b), assume the elastic forces, W_1, W_2, W_3, \dots , applied in a positive or negative direction, and draw for these forces the elastic line. The elastic line of a beam is obtained by considering the moment area as a load and drawing the funicular and string polygons. (Fig. 85 (c) and (e)). From the shape of the elastic line the following important relations are obtained: In the unloaded spans the points of application of the forces, W , lie on the third points (Fig. 85 (b), centers of gravity of triangles); the intersection, b_2 , of the component of the outer lines of the bending diagram (which touch W_1 and W_2) lies on a perpendicular which is derived by interchanging the third line, $\frac{l_1}{3}$ and $\frac{l_2}{3}$, of the respective spans (Fig. 85(c)). This perpendicular is called the reversed third line. In the same manner, W_4 and W_5 intersect in the point, c_2 . Thus, the different angles or corners of the bending diagrams lie on the third lines except the corner at W_3 , and

* "Taschenbuch für Bauingenieur," E. H. Max Foerster, Fourth Edition, pp. 361-363; "Anwendungen der Graphischen Statik," C. Culmann; "Der Kontinuierliche Balken," W. Ritter, 1900.

the components of W_1 , W_2 , W_3 , W_4 , and W_5 , fall in the reversed third lines no matter how the second span is loaded.

2.—Fig. 85 (d) shows the left-hand part of the bending diagrams enlarged from Fig. 85 (c). The corners of the triangle, $1-2-b_2$, lie each on a fixed perpendicular and two of its sides run through fixed points; therefore, the third side must also go through a certain fixed point that is in the same relation of the two other fixed points. This condition holds for the other spans and thus it can be proved that in every span of a continuous beam there are in the span line two fixed points, i_1 and k_2 , which depend only on the relation of the spans to each other, and not on the loading of the spans.

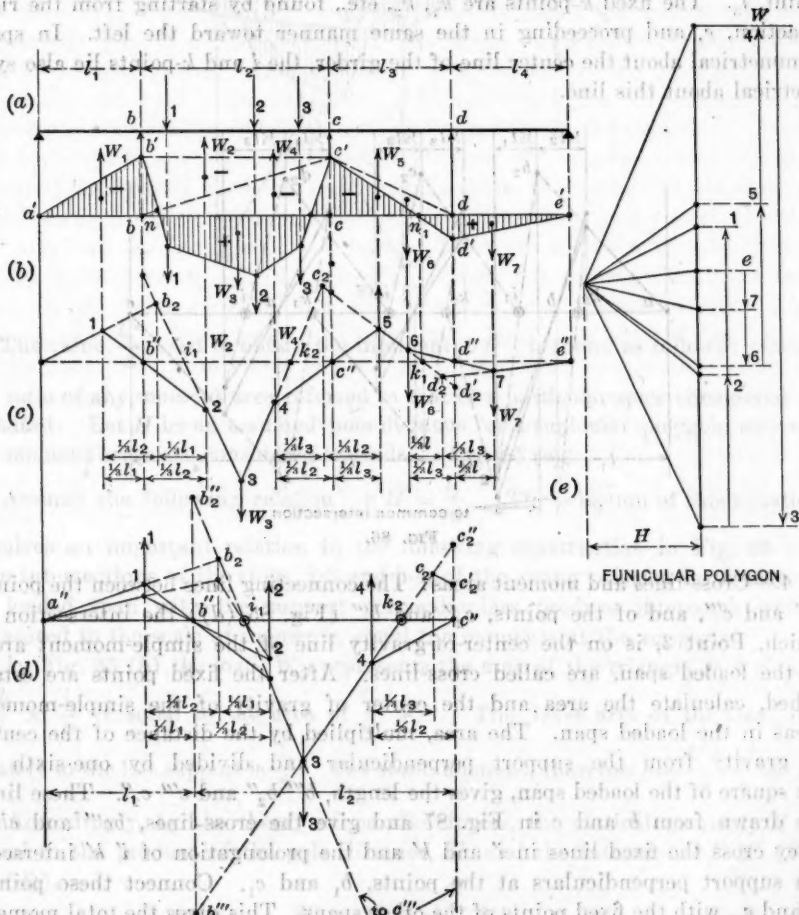


Fig. 85.

The moment in any point, i_1 , is zero for all spans to the right, and the moment in any point, k_2 , is zero for all spans to the left. The left support, a'' , is thereby the i or fixed left point for the first span; the right support is the k or fixed point for the last span. Thus, in the indicated perpendiculars

running through the fixed points (the fixed lines), lie the zero moment points and the turning points of the bending diagram.

Fig. 3.—The fixed points (Fig. 86): Draw the third lines in all spans and from the left support, a , draw any inclined line by which the second third line of span, l_1 , and the reversed third line of the second span are intersected at the points, 1 and b_2 . Then draw any line through the points, 1 and b , and produce it to its intersection (Point 2) with the first third line of the second span. The intersection of the line connecting the points, 2 and b_2 , with the horizontal will establish i_1 the first fixed i -point of the second span. Starting, now, from i_1 repeat the operation and establish the point, i_2 , and thence the point, i_3 . The fixed k -points are k_1 , k_2 , etc., found by starting from the right reaction, e , and proceeding in the same manner toward the left. In spans symmetrical about the center line of the girder, the i and k -points lie also symmetrical about this line.

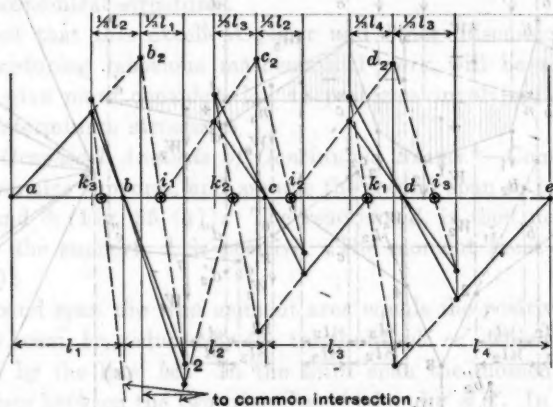


FIG. 86.

4.—Cross-lines and moment areas: The connecting lines between the points, b_2'' and c''' , and of the points, c_2'' and b''' (Fig. 85 (d)) the intersection of which, Point 3, is on the center-of-gravity line of the simple-moment areas of the loaded span, are called cross-lines. After the fixed points are established, calculate the area and the center of gravity of the simple-moment areas in the loaded span. The area, multiplied by the distance of the center of gravity from the support perpendicular and divided by one-sixth of the square of the loaded span, gives the length, $b''' b_2''$ and $c''' c_2''$. These lines are drawn from b and c in Fig. 87 and give the cross-lines, bc''' and cb''' . They cross the fixed lines in i' and k' and the prolongation of $i' k'$ intersects the support perpendiculars at the points, b_1 and c_1 . Connect these points, b_1 and c_1 , with the fixed points of the other spans. This gives the total moment areas for the continuous beam.

For a single concentration or moving load, P , in the second span (Fig. 83), construct the simple beam moment diagram, bSc . The area of triangle, bSc , equals $\frac{h l_2}{2}$. Its center of gravity is $\frac{1}{3} (l_2 + x)$ from the support vertical,

$b b''$. The static moment of this area about b is $\frac{1}{6} l_2 h (l_2 + x)$. Dividing this static moment by $\frac{l_2^2}{6}$, the value of the intercept, $b b''$, is found to be $\frac{h}{l_2} (l_2 + x)$.

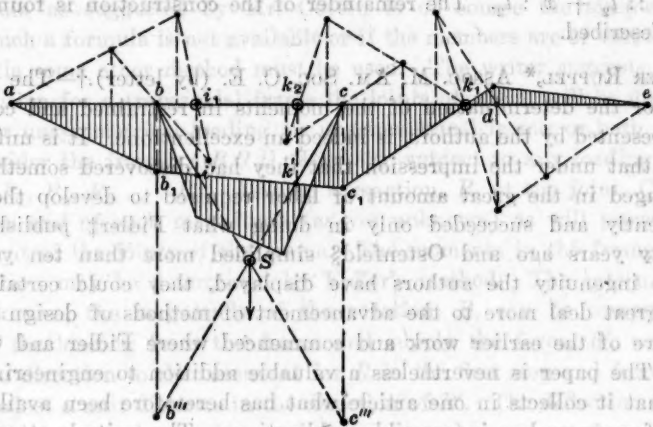


FIG. 87.

The value, $\frac{l_2^2}{6}$, used to obtain the intercept, $b b''$, is found as follows: Call r the ratio of any moment area referred to that of a particular span considered as standard. Let H be an assumed pole distance for a funicular polygon wherein the moment areas are considered as loads. (Fig. 85 (e)).

Assume the following relation: $r H = \frac{l_2^2}{6}$. The selection of this equation involves an important relation in the following construction in Fig. 85 (c). The intersections of the sides, 2-3 and 3-4, of the string polygon in the second or loaded span with the support perpendiculars produce intercepts which, measured to the scale of moments, equal the moments at the supports.

In Fig. 85 (b) the load, W_2 , represents the area of the triangle, $b' b c'$, and $\frac{b' b}{2} \times \frac{l_2}{r}$ is equal to the area of $b' b c'$. The lever arm of the load, W_2 , relative to the left support is $\frac{l_2}{3}$. The static moment, therefore, $= \frac{b' b l_2^2}{6} = M$.

Accordingly, by the theory of parallel forces, the statical moment, M , is equal to the product of the pole distance, H , and the intercept, $b_2'' b''$ (Fig. 85 (d)),

$M = H \times b_2'' b''$. Equating the two expressions and substituting $r H$ for $\frac{l_2^2}{6}$, the following relation is obtained:

$$H \times b_2'' b'' = \frac{b' b l_2^2}{6} \times \frac{1}{r}$$

or, $b_2'' b''$ in Fig. 85 (d) is equal to $b' b$ in Fig. 85 (b) and, likewise, $c_2'' c''$ in Fig. 85 (d) is equal to $c' c$ in Fig. 85 (b).

The intercept can be easily found graphically by laying off in Fig. 83 the distance, l_2 , to the right of the point of concentration giving the point, n , and drawing the line in nSb'' . By similar triangles is obtained the relation, $b b'' : h :: l_2 + x : l_2$. The remainder of the construction is found as previously described.

WALTER RUPPEL,* Assoc. M. Am. Soc. C. E. (by letter).†—The graphical method for the determination of the moments in restrained and continuous beams, presented by the authors, is indeed an excellent one. It is unfortunate, however, that under the impression that they had discovered something new, they engaged in the great amount of labor required to develop the method independently and succeeded only in doing what Fidler‡ published more than forty years ago and Ostenfeld§ simplified more than ten years ago. With the ingenuity the authors have displayed, they could certainly have added a great deal more to the advancement of methods of design had they been aware of the earlier work and commenced where Fidler and Ostenfeld left off. The paper is nevertheless a valuable addition to engineering literature in that it collects in one article what has heretofore been available in a number of more or less inaccessible publications. The writer's attention was first called to the method by the paper || by F. E. Richart, Assoc. M. Am. Soc. C. E., and W. M. Wilson, M. Am. Soc. C. E., and he has used it to good advantage since then, particularly in a form extended to the treatment of continuous beams of varying moment of inertia, in the design of parts of the Estuary Subway, Oakland, Calif. The writer will not dwell on the similarity of the authors' method with the earlier ones. That has been well covered by Mr. Richart¶ and S. M. Cotten,** Assoc. M. Am. Soc. C. E., in their discussions in which, in addition, they point out a number of geometric conceptions which clarify the method remarkably.

SOLUTION OF FRAMES

Trapezoidal Frame.—The authors touch lightly on the solution of frames in which there is a displacement of the joints such as would occur in the structure shown in Fig. 16†† if subjected to a horizontal load. The "small secondary correction" of which they speak may be greater than the stresses before correction, if, for example, the horizontal load were acting alone at one of the upper joints. The only stresses then, neglecting dead load, are those due to the displacement of the joints and there is no limit to their possible magnitude. The authors refer the reader to Section 17‡‡ but there they fail to state in what

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† Received by the Secretary, January 26, 1926.

‡ *Minutes of Proceedings*, Inst. C. E., Vol. LXXIV, 1883, p. 196, and "A Practical Treatise on Bridge Construction," by T. Claxton Fidler.

§ "Teknisk Statik," Vol. 2, Second Edition, by A. Ostenfeld, Copenhagen, Denmark.

|| *Engineering and Contracting*, June 23, 1920.

¶ *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 2085.

** *Loc. cit.*, March, 1926, Papers and Discussions, p. 484.

†† *Loc. cit.*, October, 1925, Papers and Discussions, p. 1606.

‡‡ *Loc. cit.*, p. 1623.

manner this problem may be solved since the deflection of the joints is unknown. This problem puzzled the writer for some time and he has not been alone in the quandary. Possibly the authors will explain their method if it is simpler or different from the one suggested here.

If the members of the frame are of uniform moment of inertia the use of a formula as suggested by Mr. Cotten is of course the simplest method, but if such a formula is not available or if the members are of varying moment of inertia some other method must be used. The writer suggests the following solution for a trapezoidal frame the joints of which will be displaced due to either unsymmetrical loading, or unsymmetrical frame, or both.

Consider the frame, $A B C D$ (Fig. 88), subject to any loading due to the forces, $P_1, P_2, P_3 \dots P_n$. Assume a reaction, R , at the joint, C , of known direction and of such magnitude (as yet unknown) as will prevent the displacement of the joints of the frame. The moments in the frame under this assumption may be determined by Fidler's method. The internal moments being known, the magnitude of the reaction, R , can be computed. Now, remove the loading from the frame and supply the force, R' , acting in the opposite direction to that assumed for R in the first step and force the point, C , to deflect a unit distance in the direction of R' . The deflection of B and C , perpendicular to the various members, may be computed or determined graphically. The moments can now be found by Fidler's method as applied to the settlement of supports as explained by Mr. Cotten. The computer is warned to consider the signs of the deflections properly. Now, determine the magnitude of R' in the same manner as for R . It is evident that the true

displacement of the point, C , is equal to $\frac{R}{R'}$ times the assumed deflection due to R' and that the true moments in the frame are equal to those found in the first step plus $\frac{R}{R'}$ times those found in the second step; all with due regard to signs.

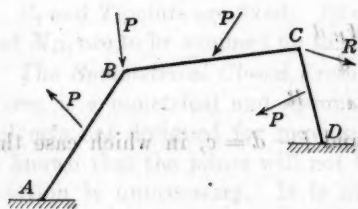


FIG. 88.

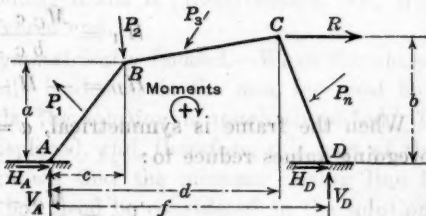


FIG. 89.

This process needs no proof; it should be practically self-evident. What has actually been done is to apply two equal and opposite loads at one of the upper panel points, applying first one with the actual loading on the frame and then the other acting alone, to solve the two cases independently and to add the stresses produced. The stresses due to the equal and opposite loads at the upper joint cancel, leaving the true stresses.

In order to avoid the labor of solving the equations necessary to determine R and R' in each problem, the writer has solved these equations in general terms so that the reactions at the legs, and R can be evaluated easily by substitution. It is to be noted that in the general case following, that A and D need not be on the same level. The horizontal and vertical components of the reactions at A and B can be determined by resolving the components, H_A , V_A , and H_B , V_B , into others of any desired direction either graphically or analytically. (Fig. 89.)

Assume R to be parallel to the line connecting A and D .

Let H = the component, parallel to AD , of all external loads on the frame.

V = the component perpendicular to AD , of all external loads on the frame.

M_B = the summation of the external and internal moments about and to the left of B , except those due to the reactions, H_A and V_A .

M_C = the summation of the external and internal moments about and to the left of C , except those due to the reactions, H_A and V_A .

M_D = the summation of the external and internal moments about and to the left of D , except those due to the reactions, H_A , V_A , and R .

The equations of equilibrium are:

$$H_A + H_D + H + R = 0$$

$$V_A + V_D + V = 0$$

$$M_B + V_A c - H_A a = 0$$

$$M_C + V_A d - H_A b = 0$$

$$M_D + V_A f + R b = 0$$

and the solution for the general case is:

$$V_A = \frac{M_C a - M_B b}{b c - a d}$$

$$V_D = -V_A - V$$

$$R = \frac{-V_A f - M_D}{b}$$

$$H_A = \frac{M_C c - M_B d}{b c - a d}$$

$$H_D = -H - H_A - R$$

When the frame is symmetrical, $a = b$ and $f = d = c$, in which case the foregoing values reduce to:

$$V_A = \frac{M_C - M_B}{c - d}$$

$$V_D = -V_A - V$$

$$R = \frac{-V_A f - M_D}{a}$$

$$H_A = \frac{M_C c - M_B d}{a(c - d)}$$

$$H_D = -H - H_A - R$$

When the frame is rectangular, $a = b = d$, and $c = 0$, in which case the values still further simplify to:

$$V_A = \frac{M_B - M_C}{d}$$

$$V_D = -V_A - V$$

$$R = \frac{M_C - M_B - M_D}{a}$$

$$H_A = \frac{M_B}{a}$$

$$H_D = -H - H_A - R$$

The Unsymmetrical Closed Frame.—The method outlined is also applicable to the unsymmetrical closed frame of four members under unsymmetrical loading. The solution is identical, except that Ostenfeld's auxiliary diagram must be solved by trial since no point on the moment closing line is known from which to start the construction. Let AD be the fourth member of the frame (otherwise similar to Figs 88 and 89) and assume that the points, A and D , are not displaced. Actually they may move in space but it may be assumed that any pair of points do not move with respect to each other. The reactions, H_A , V_A , and H_D , V_D , may not actually exist. However, they must be supplied in the two solutions, but if the external loads balance, which they must do if there are no reactions and transposition is not to take place, their values in the two solutions will be equal and opposite.

The moment closing line is determined as follows: Make a cut at Joint D and lay the frame out as a continuous beam. Assume a moment at the one severed end and draw the moment closing line. It is known that the moments at the two severed ends must be equal. If they do not come out equal in the construction, the assumed value was in error. Adjust it and try again. The second or third trial should be successful. It is to be noted that only the P -points (see Fig. 92) must be found anew for each trial, the U , V , and T -points are fixed. In computing R and R' , the moments, M_B , M_C , and M_D , are to be summed up to the severed end.

The Symmetrical Closed Frame, Symmetrically Loaded.—When the closed frame is symmetrical and symmetrically loaded as is the case for most box culverts not designed for moving loads, the solution is much simplified. It is known that the joints will not be displaced, and, therefore, this part of the solution is unnecessary. It is also known that the moment closing line is symmetrical so that only one-half the box need be considered in the solution. The determination of the moment closing line by trial is aided by the fact that it is parallel to the neutral axis in two of the members.

HAUGHED BEAMS

Introduction.—The authors have shown how symmetrical beams of varying moment of inertia may be treated by Fidler's method and Mr. Cotten in his discussion, has explained how both the symmetrical and the unsymmetrical case may be solved. In continuous systems, haunched beams have numerous

advantages over beams of uniform moment of inertia. These advantages are mainly economic and esthetic.

Effect of Haunches on Bending Moments.—In continuous beams of uniform moment of inertia, the cross-section is usually determined by the bending moment and the shear at the support. When no haunches are provided there is a considerable waste of material at the center. Realizing this, it has often been the practice to reduce the section at the center by haunching the beam. Usually, however, it has not been the custom to take into account the effect of the variation of moment of inertia on the distribution of the bending moments. By most methods it is difficult to determine the bending moments in beams of varying moment of inertia and it has not been realized generally that there are any great changes in the bending moments due to this variation.

On a restrained beam haunches decrease the center bending moment and increase the bending moment at the support as compared with a beam of constant cross-section. In some instances the difference may become quite large. Consider a beam under uniform load, having fixed ends, straight haunches one-fourth the length of the span, and a depth at the supports one and one-half times that of the uniform portion. The center moment is only 74% of that for a beam of uniform section, whereas the end moment is 13% greater. The writer has an actual case in mind, namely, that of a reinforced concrete retaining wall* about 30 ft. high, continuous at the base with a floor-slab and simply supported at the top. The effect of tapering the wall from 5 ft. at the base to 2 ft. at the top, was to increase the moment at the base by 25% over that for a wall of uniform thickness, but still effecting a considerable saving in the upper part of the wall. Had the base been completely fixed the increase in moment would have been almost 33 per cent. Although neither of these structures would have failed had the increased moment due to the haunch been disregarded, it is readily seen that the actual factor of safety may be considerably less than was intended and yet there is a waste of material at other points in the structure. The action of continuous haunched beams, the effect on the distribution of moments, and the savings in an approach trestle to the Pulaski Bridge over the Salmon River at Pulaski, N. Y., has been described† by E. H. Harder, Assoc. M. Am. Soc. C. E.

Advantages of Haunched Beams.—Haunches better accommodate the beam to its shearing stresses and in a concrete beam often eliminate the necessity of extremely heavy shear reinforcement. The added depth at the support also decreases the bond stresses, making the use of larger bars possible. The neutral axis of a haunched beam is actually curved and the induced thrust provides added safety. The ability to carry shear is also increased due to this inclination.‡ Another advantage of haunches lies in the reduction of the high stresses existing at sharp re-entrant angles as at the corners of a box culvert. Theoretically, it has been shown and tests have proven that extremely

* Outside substructure wall, Alameda Portal Building, Estuary Subway, Oakland, Calif.

† "Building a Concrete Arch Around an Old Iron Bridge," *Engineering News-Record*, Vol. 92, No. 8, February 21, 1924.

‡ See "Der Eisenbetonbau," by Prof. Dr. E. Mörsch; Fifth Edition, Vol. I, Pt. 2, p. 16, and also "Concrete Engineer's Handbook," by Hool and Johnson, p. 314.

high stresses occur at sharp re-entrant angles due to the abrupt curvature of the neutral axis at such points.*

Saving in Material.—The possible saving in material due to haunches varies considerably with the loading and the conditions of restraint. In the beam referred to previously with a depth of haunch one and one-half times that at the center, the saving in material is about 10 per cent. For this loading and condition of restraint the most economical depth of haunch is about twice the center depth for which the saving in material is about 14% over a uniform beam. There is an additional saving due to the reduction in the dead load of the beam itself and due to the concentration of the material in the beam nearer to the supports.

Increased Costs Due to Haunches.—Against these savings must be balanced the increased cost of formwork, if the structural material is concrete, or the increased cost of fabrication, if it is steel. In the case of concrete the increase in form cost due to a straight haunch is small. In steel there is usually no economy until built-up members must be used.

Improved Methods of Design.—For reasons of economy, the advantages of haunched beams were earlier recognized in Europe than in the United States, mainly because of the difference in the relation of cost of labor to that of material. In Europe, principally in Germany, the increasing use of haunched beams led to the development of shorter methods for their design and also to the preparation of tables to expedite the computations. Such a set of tables has been prepared by A. Strassner.† These tables are directly applicable to Strassner's own method for designing continuous structures, a semi-analytical-graphical method of very general application, but probably not as simple as Fidler's method for conditions to which the latter is applicable. Strassner's tables give coefficients from which the angular change of the elastic curve at the supports of the simply supported beam may be determined, when subjected to the moment, 1, at the support or to certain classes of loading. When these tables were first brought to the writer's attention about two years ago, he developed a number of simple formulas by which the values in Strassner's tables could be converted so as to be directly applicable to Fidler's method and to the slope deflection method,‡ when this method is extended to beams of variable moment of inertia. With the aid of Strassner's tables and these formulas the writer has now computed a new set of tables which give coefficients directly useful in simplifying Fidler's method. These tables (Tables 6 to 32, inclusive) are presented as a part of this discussion (see Appendix). In part, these new tables are similar to, but much more extensive than, the table presented by the authors (Table 2§).

Strassner computed his tables for three types of haunch which have been referred to in the recomputed tables as Case I, II, and III. These three types,

* See "Analysis and Tests of Rigidly Connected Reinforced Concrete Beams," by Mikishi Abe, M. Am. Soc. C. E., *Bulletin 107*, Eng. Experiment Station, Univ. of Illinois, p. 99; "Elastizität und Festigkeit," by C. v. Bach; and "Drang und Zwang, Eine Höhere Festigkeitslehre für Ingenieure," by Dr.-Ing. Aug. Föppl and Dr. Ludwig Föppl, Vol. I, Second Edition, p. 248, Berlin, 1924.

† "Neuere Methoden," Vol. I, Second Edition, Berlin, 1921.

‡ *Bulletin 108*, Eng., Experiment Station, Univ. of Illinois.

§ *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1618.

together with a fourth, introduced later in this discussion, will either cover or closely approximate almost any shape that is likely to be found in practice.

Nomenclature.—In the nomenclature to be used as far as practicable the authors' terms have been followed. Others are as used by Mr. Cotten in his discussion. Some additional terms have been introduced by the writer.

Let,

I = minimum moment of inertia of the member.

I' = maximum moment of inertia of the member. (at the support).

I_z = moment of inertia of the haunch at the distance, z , from the beginning of the haunch.

h_z = depth of haunch corresponding to I_z .

a = ratio of length of haunch to span of beam.

$b = \frac{I'}{I}$.

c = an exponent indicative of the degree of curvature of the haunch in Case IV.

E = modulus of elasticity.

W = any concentrated load, the total uniform load, or the total triangular load on the member.

l = span of member.

$u = \frac{x'}{l}$ = the distance from the left support to the left characteristic point when $l = 1$.

$v = 1 - \frac{x''}{l}$ = distance from the right support to the right characteristic point when $l = 1$.

$p = A' EI$ = area of the $\frac{M}{EI}$ -diagram for $M = 1$ at the left support of the beam of Span 1 having E and $I = 1$.

$q = A'' EI$ = area of the $\frac{M}{EI}$ -diagram for $M = 1$ at the right support of the beam of Span 1 having E and $I = 1$.

$s = \frac{U}{lW}$ = height of the left characteristic point when l and $W = 1$.

$t = \frac{V}{lW}$ = height of the right characteristic point when l and $W = 1$.

Shapes of Haunches.—

Case I.—Sharply Curved Haunch (Fig. 90).—

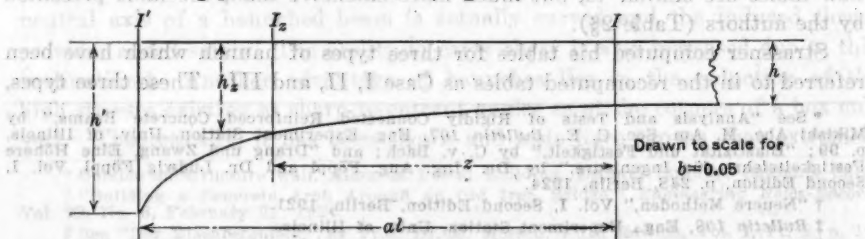


FIG. 90.

Use of the Tables.—In order to determine the moments in a continuous beam of varying or uniform moment of inertia, it has been shown by the authors and by Mr. Cotton in his discussion, that the position of several points and lines must be determined namely the height and lateral position of the V and T points (Fig. 90) and the lateral position of the T-line (one of the lines of the V-line).

As shown in Fig. 90, this haunch curves at an increasingly rapid rate as the support is reached. It is not quite tangent to the uniform part of the beam, but in practice may be made so if desired. The preceding equations are given so that the shape may be reproduced or be fitted to a given haunch.

Case II.—Straight Haunch.—This type of haunch decreases uniformly from the support to a point toward the center of the span.

Case III.—Parabolic Haunch.—This is a parabola of the second degree, the vertex is at the beginning of the haunch and the curve is tangent to the uniform part of the beam at the vertex.

Case IV.—Sharply Curved Haunches of Varying Degrees of Sharpness (Fig. 91).—Professor Max Ritter,* one of the first to develop simpler methods for treating beams of variable moment of inertia, introduced a formula for the variation of the moment of inertia, which, when substituted in the $\frac{M}{EI}$ equation, gave functions that could be readily integrated. It is the general formula of which Case I is the special condition for the value of the exponent, c , equal to 1. (See Fig. 91.)

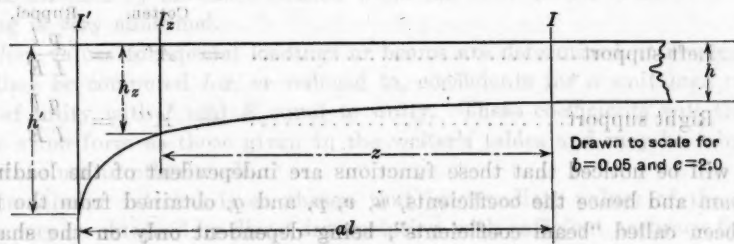


FIG. 91.

The exponent, c , which determines the degree of sharpness of the curvature of the haunch usually has a value between 0.5 and 5.0 in practical problems and by varying it between these limits, the haunch can be adapted to a great variety of shapes. As c increases, the haunch curves more sharply at the support.

* Schweizerische Bauzeitung, Vol. LIII, Nos. 18 and 19, 1914.

Use of the Tables.—In order to determine the moments in a continuous beam of varying or uniform moment of inertia, it has been shown by the authors and by Mr. Cotten in his discussion, that the position of several points and lines must be determined, namely, the height and lateral position of the U and V -points (Fidler's characteristic points) and the lateral position of the T -lines (one of the lines of Ostenfeld's auxiliary diagram).

For the four shapes of haunch (Cases I to IV), these points and lines may be located with the aid of Tables 6 to 32, inclusive. As shown by the diagrams at the head of the various tables, they are applicable to the symmetrical beam with haunches at both supports and also to the beam with a haunch at only one support.

The lateral position of the U and V -points is determined from one of Tables 6 to 11, inclusive, and Table 28, with the aid of the following relation:

$$\text{Distance of the } U\text{-line from the left support} = u l.$$

$$\text{Distance of the } V\text{-line from the right support} = v l.$$

As shown in Mr. Cotten's discussion, to determine the lateral position of the T -line it is necessary to know the area of the $\frac{M}{EI}$ -diagram when the moment, 1, is applied successively at the left and right support. This is also found by using the coefficients given in Tables 6 to 11, inclusive, and Table 28, in the following formulas: The area of the $\frac{M}{EI}$ -diagram for M equals 1 at the:

	Cotten.	Ruppel.
Left support.....	$= A' =$	$\frac{p l}{I E}$
Right support.....	$= A'' =$	$\frac{q l}{I E}$

It will be noticed that these functions are independent of the loading on the beam and hence the coefficients, u , v , p , and q , obtained from the tables have been called "beam coefficients", being dependent only on the shape of the beam.

The height of the U and V -points is dependent on the loading as well as on the shape of the beam. Coefficients for a variety of loadings have been computed and are tabulated under the heading of "Load Coefficients" in Tables 12 to 27, inclusive, and Table 29. The heights of the U and V -points are obtained by substitution in the following formulas:

$$U = \Sigma (W s) l; \text{ and } V = \Sigma (W t) l$$

The Σ -sign is used to indicate that the load must be divided into parts to which the tables for "load coefficients" are applicable.

Tables 12 to 17, inclusive, give the values of the "load coefficients", s and t , for a concentration at any twelfth point of the span. Tables 18 to 23, inclusive, and Table 29 give the values of the coefficients for a uniform load over the entire span. Tables 24 to 27, inclusive, give the coefficients for a triangular load over the whole span. Coefficients for other loadings, such as

partial, uniform, or triangular loads, etc., may be found approximately within 1% by dividing the load into a number of concentrations and then using Tables 12 to 17, inclusive, or exactly as follows: Multiply the coefficients for a unit concentration at each twelfth point by the intensity of the actual load at that point; plot these products at their respective twelfth points and draw a curve through them, as shown in Fig. 93 or Fig. 94. The area under the curve times 12, divided by the product of the span times the load considered, is the coefficient for that load. The factor, 12, is introduced to take care of the horizontal scale of the diagram. The area under the curve may be obtained by planimeter or other method. Simpson's rule or any one of Cotes' formulas* will give the area rapidly and accurately.

For Case IV, "load coefficients" are given only for a uniform load over the whole span. The coefficients for other types of loading may be found with considerable accuracy by using the values given for Case I(a) and multiplying them by the ratio of the distances from the support to the characteristic point in Case IV to that for Case I(a). This relation is exactly true for a uniform load and approximately true for other loadings, departing more from the truth as the loading ceases to approximate a uniform one and as the values of b and c depart from unity. For $b = 0.03$ and $c = 5.0$, the error in the coefficient for a central concentration is 2.1% and will probably not exceed this figure for any practical beam and combination of loading. The approximate method is certainly exact enough for all preliminary designs and may be checked by an exact method when the final design is adopted if the loading is very abnormal.

When values for special loadings or beams are determined, it is suggested that they be computed for, or reduced to, coefficients for a unit load on the span of unity with I and E equal to unity. These coefficients will then be in the same form as those given in the writer's tables and may be tabulated for future use in other work.

The tabular interval is so chosen that intermediate values of the coefficients may be obtained by direct interpolation with sufficient accuracy for any practical problem. If greater accuracy is desired, a curve may be drawn through several values of the coefficient on either side of the desired one and the value read from the curve.

Tables 24 to 27, inclusive, of triangular load coefficients are new; their equivalent does not appear in Strassner's tables. Tables 28 and 29 are computed by the writer from formulas developed with the aid of the work of Max Ritter previously referred to.

Three other tables which the writer has found useful as an aid in the solution of continuous beams of uniform moment of inertia have been added. Table 30 gives the "load coefficients" for a partial uniform load in various positions on the span. This table was originally prepared by Mr. Cotten and has been published† in a slightly different form. It is here included in the revised form at the request of Mr. Cotten.

* See "Methods of Approximate Integration", by Willis Whited, M. Am. Soc. C. E., *Engineering News*, Vol. 73, No. 17, April 29, 1915.

† *Engineering and Contracting*, Vol. 43, No. 15, April 14, 1915.

Table 31 gives load coefficients for triangular loads, with a maximum intensity at the support and extending over various parts of the span. Table 32 gives load coefficients for trapezoidal loads and for uniform and triangular loads at the upper and lower limit. In addition are given coefficients for plotting the simple beam moment diagram for these loadings. Tables 31 and 32 are particularly useful as an aid in the application of Fidler's method to structures under hydraulic load or its equivalent.

Accuracy of the Tables.—Due to the limitations of Strassner's original tables and to the mathematical operations of the recomputations by the writer, the tabular values will be found to have slight errors occasionally in the last figure. In Tables 12 to 18, inclusive, this may amount to two or three units in the last place. In the other tables, the error will usually not exceed one unit in the last place and will seldom be that large. The tables have been carefully checked to detect errors in both the original tables and in the writer's computations. All values have been plotted in two or more curves and, for many, first differences have been plotted in addition. A large number of values were computed from first principles to check points that failed to plot in a smooth curve and also to detect constant errors which the curves might not reveal. It is believed and hoped that if any errors remain, they are small and inconsequential.

The preparation of the tables has involved a considerable amount of labor and the writer hopes that the work may be of benefit to other engineers. With the aid of these tables the solution of a haunched beam involves little if any additional work over that required for a beam of uniform moment of inertia. They should lead to the more general use of haunched beams.

Application of the Elastic Theory to Haunched Concrete Beams.—Some may doubt whether the ordinary elastic theory is applicable to reinforced concrete haunched beams in which the moment of inertia and modulus of elasticity are open to question. However, if continuous concrete beams of uniform moment of inertia can be proportioned according to this theory, it may also be used for beams of variable moment of inertia. Tests have shown that if the moment of inertia of a concrete beam be computed using the effective depth and if the steel be proportioned in accordance with the computed moments, the actual stresses will be closely in accord with the theoretical ones.* It is true that occasionally tests have shown wide deviations of the actual stresses from the theoretical ones, but most of the evidence seems to point to the fact that the ordinary elastic theory is applicable, within safe limits, to reinforced concrete structures when the steel is properly proportioned and care is taken to obtain a uniform concrete throughout the structure. The latter requires not only uniform proportions of cement and aggregate, but proper mixing, uniform water content, and uniformity of curing as well.

Illustrative Example.—It now remains to solve an actual problem to illustrate the use of the tables. For comparison, consider the beam shown by

* For further discussion of this subject and for the results of a large number of independent tests reference is made to, "Der Eisenbetonbau," by Prof. E. Mörsch, Third Edition, p. 210; "Neuere Methoden," by A. Strassner, Vol. I, Second Edition, p. 73, and "Analysis and Tests of Rigidly Connected Reinforced Concrete Frames," by Mikishl Abe, M. Am. Soc. C. E., Bulletin 107, Eng. Experiment Station, Univ. of Illinois.

the authors in Fig. 23,* but add an unsymmetrical beam loaded as shown in Fig. 92, continuous at the left end and simply supported at the right end. This will completely illustrate the problem and the tables.

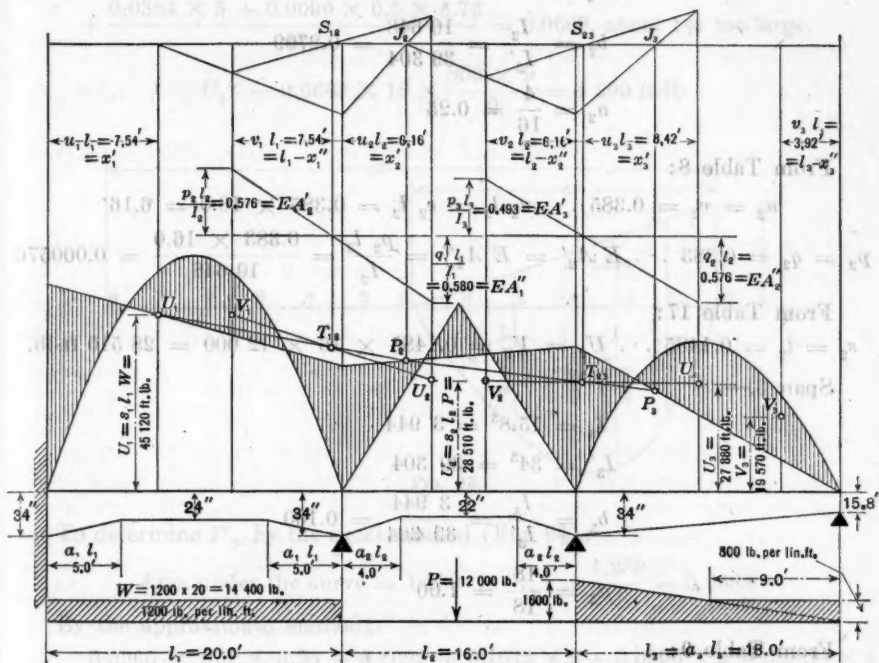


FIG. 92.

It is to be noted that the method used for determining the lateral position of the T -line, in Fig. 92, is the graphical solution of the equations given by Mr. Cotten in his solution of the general case. The computed values of $E A'$ and $E A''$ have been multiplied by 1000 for use in this diagram.

Span 1.—

$$I_1' = 34^3 = 39\,304$$

$$I_1 = 24^3 = 13\,824$$

$$b_1 = \frac{I_1}{I_1'} = \frac{13\,824}{39\,304} = 0.3517$$

$$a_1 = \frac{5}{20} = 0.25$$

From Table 8:

$$u_1 = v_1 = 0.377 \quad u_1 l_1 = v_1 l_1 = 0.377 \times 20.0 = 7.54$$

$$p_1 = q_1 = 0.401 \quad E A_1' = E A_1'' = \frac{p_1 l_1}{I_1} = \frac{0.401 \times 20.0}{13\,824} = 0.000580$$

From Table 20:

$$s_1 = t_1 = 0.0940 \quad U_1 = V_1 = 0.0940 \times 20 \times 20 \times 1\,200 = 45\,120 \text{ ft.-lb.}$$

Span 2.—

$$I_2 = 34^3 = 39\,304$$

$$I_2 = 22^3 = 10\,648$$

$$b_2 = \frac{I_2}{I_2'} = \frac{10\,648}{39\,304} = 0.2709$$

$$a_2 = \frac{4}{16} = 0.25$$

From Table 8:

$$u_2 = v_2 = 0.385 \therefore u_2 l_2 = v_2 l_2 = 0.385 \times 16.0 = 6.16'$$

$$p_2 = q_2 = 0.383 \therefore E A_2' = E A_2'' = \frac{p_2 l_2}{I_2} = \frac{0.383 \times 16.0}{10\,648} = 0.000576$$

From Table 17:

$$s_2 = t_2 = 0.1485 \therefore U_2 = V_2 = 0.1485 \times 16 \times 12\,000 = 28\,510 \text{ ft.-lb.}$$

Span 3.—

$$I_3 = 15.8^3 = 3\,944$$

$$I_3' = 34^3 = 39\,304$$

$$b_3 = \frac{I_3}{I_3'} = \frac{3\,944}{39\,304} = 0.100$$

$$a_3 = \frac{18}{18} = 1.00$$

From Table 9:

$$u_3 = 0.468 \therefore u_3 l_3 = 0.468 \times 18.0 = 8.42'$$

$$v_3 = 0.218 \therefore v_3 l_3 = 0.218 \times 18.0 = 3.92'$$

$$p_3 = 0.108 \therefore E A_3' = \frac{p_3 l_3}{I_3} = \frac{0.108 \times 18.0}{3\,944} = 0.000493$$

$$q_3 = 0.232 \therefore E A_3'' = \frac{q_3 l_3}{I_3} = \frac{0.232 \times 18.0}{3\,944} = 0.001059$$

From Table 26, for the total triangular load:

$$s_3' = 0.0910 \therefore U_3' = 0.0910 \times 18 \times \frac{1\,600 \times 18}{2} = 23\,590 \text{ ft.-lb.}$$

$$t_3' = 0.0583 \therefore V_3' = 0.0583 \times 18 \times \frac{1\,600 \times 18}{2} = 15\,110 \text{ ft.-lb.}$$

To find the values of U_3'' and V_3'' for the partial triangular load it will be necessary to compute the coefficient with the aid of Table 15 according to the method previously outlined on pages 688 and 689. For convenience in computing the coefficient assume a span of 12 and a load of 18. Then to determine s_3'' by the exact method (Fig. 93):

$$\text{Area under the curve} = 1.192; \text{ hence } s_3'' = \frac{1.192}{18} = 0.0662$$

By the approximate method:

$$s_3'' = \frac{0.1367 \times 0.5 \times 0.25 + 0.1335 + 0.1213 \times 2 + 0.1007 \times 3 + 0.0726 \times 4}{18}$$

$$+ \frac{0.0384 \times 5 + 0.0096 \times 0.5 \times 5.75}{18} = 0.0669, \text{ about 1\% too large.}$$

$$U_3'' = 0.0662 \times 18 \times \frac{800 \times 9}{2} = 4\,290 \text{ ft.-lb.}$$

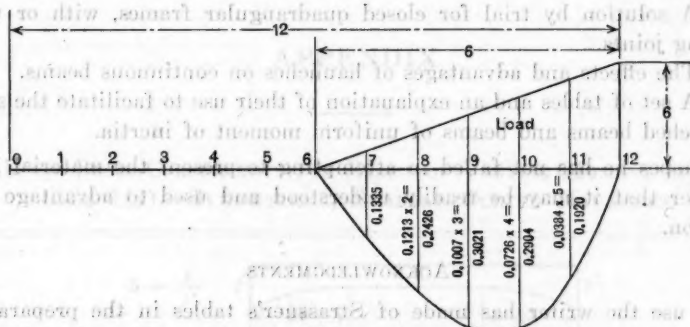


Fig. 93.

To determine t_3'' , by the exact method (Fig. 94):

$$\text{Area under the curve} = 1.239 \therefore t_3'' = \frac{1.239}{18} = 0.0688.$$

By the approximate method:

$$t_3'' = \frac{0.0965 \times 0.5 \times 0.25 + 0.1022 + 0.1042 \times 2 + 0.0990 \times 3 + 0.0829 \times 4}{18}$$

$$+ \frac{0.0520 \times 5 + 0.0130 \times 0.5 \times 5.75}{18} = 0.0694, \text{ about 1\% too large.}$$

$$V_3'' = 0.0688 \times 18 \times \frac{800 \times 9}{2} = 4\,460 \text{ ft.-lb.}$$

$$U_3 = U_3' + U_3'' = 23\,590 + 4\,290 = 27\,880 \text{ ft.-lb.}$$

$$V_3 = V_3' + V_3'' = 15\,110 + 4\,460 = 19\,570 \text{ ft.-lb.}$$

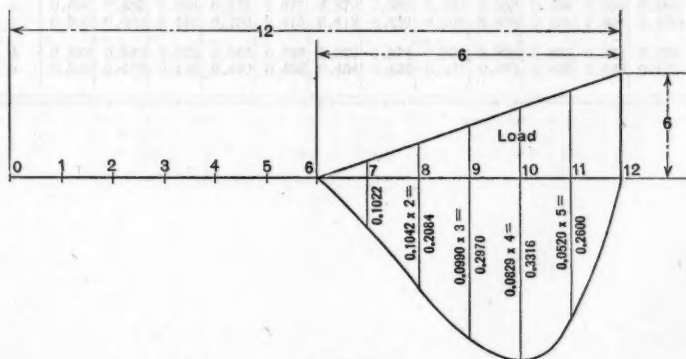


Fig. 94.

All these values are indicated in their proper positions in Fig. 92 and the moment closing line drawn as explained by the authors in their paper and by Mr. Cotten in his discussion.

CONCLUSION

The writer has endeavored to present in this discussion:

- 1.—A direct solution of trapezoidal frames with deflecting joints, by Fidler's method.
- 2.—A solution by trial for closed quadrangular frames, with or without deflecting joints.
- 3.—The effects and advantages of haunches on continuous beams.
- 4.—A set of tables and an explanation of their use to facilitate the solution of haunched beams and beams of uniform moment of inertia.

He hopes he has not failed in attempting to present the material in such a manner that it may be readily understood and used to advantage by the profession.

ACKNOWLEDGMENTS

The use the writer has made of Strassner's tables in the preparation of his own, has already been referred to. Without those tables, the labor would have been many times as great as it has been and the computation of such an extensive set of tables would probably not have been undertaken. The writer wishes to acknowledge the valuable assistance he has received from Mr. Cotten and Herman Schorer, Assoc. M. Am. Soc. C. E., to the former for suggestions and assistance in checking the text of the discussion, and to the latter for suggestions and for bringing to the writer's attention and making available many of the foreign publications.

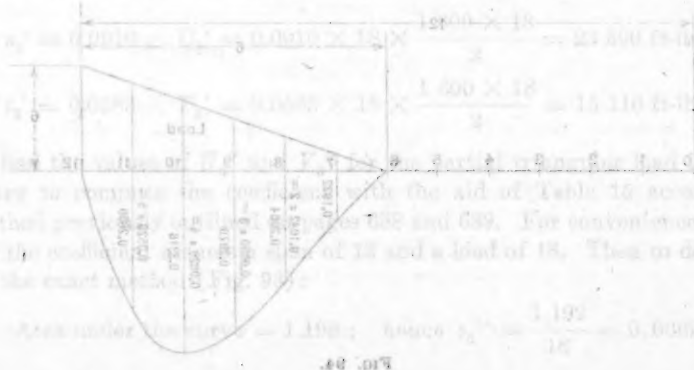


TABLE 7.—Beam Coefficients for Beams of Varying Moment of Inertia.
Case I (b).—Unsymmetrical, Sharply Curved Haunch.

APPENDIX

TABLE 6.—BEAM COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CASE I(a).—SYMMETRICAL, SHARPLY CURVED HAUNCHES.

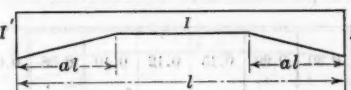
VALUES OF b		0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9	4.0	4.1	4.2	4.3	4.4	4.5	4.6	4.7	4.8	4.9	5.0	5.1	5.2	5.3	5.4	5.5	5.6	5.7	5.8	5.9	6.0	6.1	6.2	6.3	6.4	6.5	6.6	6.7	6.8	6.9	7.0	7.1	7.2	7.3	7.4	7.5	7.6	7.7	7.8	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	8.9	9.0	9.1	9.2	9.3	9.4	9.5	9.6	9.7	9.8	9.9	10.0
VALUES OF a .		VALUES OF b_c																																																																																																				
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02																																																																																								
0.50	u	0.333	0.354	0.378	0.389	0.395	0.399	0.402	0.404	0.407	0.409	0.410	0.412	0.413	v	q																																																																																						
	p	0.500	0.400	0.325	0.300	0.288	0.280	0.275	0.270	0.265	0.263	0.260	0.257	0.255	v	q																																																																																						
0.40	u	0.333	0.356	0.380	0.390	0.395	0.398	0.401	0.403	0.406	0.407	0.408	0.410	0.411	v	q																																																																																						
	p	0.500	0.420	0.360	0.340	0.330	0.324	0.320	0.316	0.312	0.310	0.308	0.306	0.304	v	q																																																																																						
0.35	u	0.333	0.356	0.379	0.388	0.393	0.396	0.398	0.400	0.403	0.404	0.405	0.406	0.407	v	q																																																																																						
	p	0.500	0.430	0.378	0.360	0.351	0.346	0.342	0.339	0.335	0.334	0.332	0.330	0.328	v	q																																																																																						
0.30	u	0.333	0.356	0.377	0.385	0.388	0.392	0.394	0.396	0.397	0.399	0.400	0.400	0.401	v	q																																																																																						
	p	0.500	0.440	0.395	0.380	0.378	0.368	0.365	0.362	0.359	0.358	0.356	0.355	0.353	v	q																																																																																						
0.25	u	0.333	0.354	0.373	0.380	0.384	0.386	0.388	0.389	0.391	0.392	0.393	0.393	0.394	v	q																																																																																						
	p	0.500	0.450	0.413	0.400	0.394	0.390	0.388	0.385	0.382	0.381	0.380	0.379	0.378	v	q																																																																																						
0.20	u	0.333	0.352	0.368	0.374	0.377	0.379	0.380	0.382	0.383	0.384	0.384	0.385	0.385	v	q																																																																																						
	p	0.500	0.460	0.430	0.420	0.415	0.412	0.410	0.408	0.406	0.405	0.404	0.403	0.402	v	q																																																																																						
0.15	u	0.333	0.349	0.362	0.366	0.369	0.370	0.371	0.372	0.373	0.373	0.374	0.374	0.375	v	q																																																																																						
	p	0.500	0.470	0.448	0.440	0.436	0.434	0.433	0.431	0.430	0.429	0.428	0.427	0.426	v	q																																																																																						

TABLE 7.—BEAM COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CASE I(b).—UNSYMMETRICAL, SHARPLY CURVED HAUNCH.

Diagram illustrating the geometry of a beam of length l with a central point load P . The beam is supported at both ends. The distance from each support to the center is $l/2$. The diagram shows the beam with a central point load P and a reaction force P at each support. The distance from the center to the supports is labeled $l/2$. The diagram is labeled "Fig. 10".

TABLE 8.—BEAM COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CASE II(a).—SYMMETRICAL, STRAIGHT HAUNCHES.

TABLE 8.—BEAM COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CASE II(a).—SYMMETRICAL, STRAIGHT HAUNCHES.

$$b = \frac{I}{I'}$$


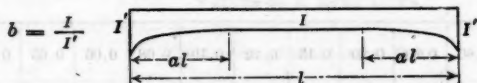
Values of a.		VALUES OF b.													
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
0.50	u	0.333	0.354	0.390	0.394	0.403	0.410	0.416	0.422	0.430	0.434	0.439	0.446	0.455	v
	p	0.500	0.389	0.279	0.232	0.203	0.184	0.170	0.154	0.136	0.126	0.115	0.102	0.086	q
0.40	u	0.333	0.358	0.387	0.402	0.412	0.420	0.425	0.431	0.440	0.444	0.449	0.455	0.462	v
	p	0.500	0.411	0.324	0.286	0.263	0.243	0.236	0.223	0.209	0.201	0.192	0.182	0.169	q
0.35	u	0.333	0.358	0.387	0.402	0.412	0.419	0.424	0.430	0.437	0.441	0.446	0.450	0.457	v
	p	0.500	0.422	0.346	0.312	0.292	0.279	0.269	0.258	0.245	0.238	0.230	0.221	0.210	q
0.30	u	0.333	0.357	0.385	0.400	0.408	0.415	0.419	0.425	0.431	0.434	0.438	0.443	0.448	v
	p	0.500	0.493	0.368	0.339	0.322	0.310	0.302	0.292	0.282	0.276	0.269	0.261	0.252	q
0.25	u	0.333	0.356	0.382	0.394	0.402	0.407	0.412	0.416	0.422	0.425	0.428	0.432	0.436	v
	p	0.500	0.444	0.390	0.366	0.352	0.342	0.335	0.327	0.318	0.313	0.307	0.301	0.293	q
0.20	u	0.333	0.354	0.376	0.387	0.393	0.398	0.401	0.405	0.409	0.412	0.415	0.418	0.422	v
	p	0.500	0.455	0.412	0.392	0.381	0.374	0.368	0.362	0.354	0.350	0.346	0.341	0.334	q
0.15	u	0.333	0.350	0.368	0.377	0.382	0.385	0.388	0.391	0.394	0.396	0.398	0.401	0.404	v
	p	0.500	0.466	0.434	0.420	0.411	0.405	0.401	0.396	0.391	0.388	0.384	0.380	0.376	q

TABLE 9.—BEAM COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CASE II(b).—UNSYMMETRICAL, STRAIGHT HAUNCH.

$b = \frac{I}{I'}$

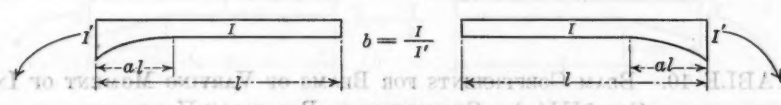
Values of a .	Haunch at left support.	VALUES OF b .												Haunch at right support.	
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
1.00	u	0.333	0.362	0.408	0.427	0.443	0.458	0.468	0.482	0.500	0.510	0.523	0.540	0.566	v
	v	0.333	0.306	0.269	0.250	0.236	0.226	0.218	0.207	0.196	0.188	0.179	0.168	0.153	u
	p	0.500	0.356	0.224	0.171	0.141	0.122	0.108	0.098	0.077	0.068	0.059	0.048	0.037	q
	q	0.500	0.422	0.335	0.292	0.266	0.247	0.232	0.215	0.196	0.184	0.171	0.156	0.136	p
0.50	u	0.333	0.373	0.425	0.455	0.475	0.490	0.501	0.515	0.531	0.540	0.551	0.564	0.580	v
	v	0.333	0.317	0.297	0.288	0.282	0.277	0.274	0.270	0.266	0.263	0.260	0.256	0.252	u
	p	0.500	0.408	0.321	0.284	0.262	0.248	0.237	0.225	0.212	0.205	0.197	0.188	0.177	q
	q	0.500	0.480	0.459	0.448	0.441	0.437	0.433	0.429	0.424	0.421	0.418	0.414	0.409	p
0.40	u	0.333	0.370	0.417	0.441	0.458	0.470	0.479	0.489	0.502	0.509	0.517	0.527	0.538	v
	v	0.333	0.321	0.308	0.301	0.297	0.294	0.292	0.289	0.286	0.284	0.282	0.280	0.277	u
	p	0.500	0.423	0.350	0.319	0.300	0.288	0.279	0.269	0.258	0.251	0.244	0.237	0.227	q
	q	0.500	0.487	0.474	0.467	0.462	0.460	0.457	0.454	0.451	0.450	0.447	0.445	0.442	p
0.35	u	0.333	0.368	0.410	0.433	0.447	0.456	0.465	0.474	0.485	0.491	0.498	0.505	0.515	v
	v	0.333	0.324	0.313	0.308	0.305	0.302	0.300	0.298	0.296	0.295	0.293	0.291	0.288	u
	p	0.500	0.432	0.366	0.338	0.321	0.310	0.302	0.293	0.283	0.277	0.271	0.264	0.255	q
	q	0.500	0.490	0.480	0.474	0.471	0.469	0.467	0.465	0.463	0.461	0.460	0.458	0.455	p
0.30	u	0.333	0.365	0.403	0.422	0.435	0.443	0.450	0.457	0.466	0.471	0.477	0.483	0.492	v
	v	0.333	0.326	0.318	0.314	0.312	0.310	0.308	0.307	0.305	0.304	0.303	0.301	0.299	u
	p	0.500	0.440	0.382	0.358	0.348	0.333	0.326	0.318	0.309	0.304	0.299	0.292	0.284	q
	q	0.500	0.493	0.485	0.481	0.479	0.477	0.476	0.474	0.473	0.472	0.470	0.469	0.467	p
0.25	u	0.333	0.362	0.394	0.410	0.420	0.427	0.433	0.439	0.446	0.450	0.455	0.460	0.466	v
	v	0.333	0.328	0.322	0.319	0.318	0.316	0.315	0.314	0.312	0.312	0.311	0.310	0.309	u
	p	0.500	0.449	0.400	0.379	0.366	0.358	0.352	0.345	0.337	0.333	0.328	0.322	0.316	q
	q	0.500	0.495	0.490	0.487	0.485	0.484	0.483	0.482	0.481	0.480	0.480	0.478	0.477	p
0.20	u	0.333	0.357	0.384	0.397	0.405	0.411	0.415	0.420	0.425	0.428	0.432	0.436	0.441	v
	v	0.333	0.330	0.326	0.324	0.323	0.322	0.321	0.321	0.319	0.319	0.318	0.318	0.317	u
	p	0.500	0.458	0.418	0.401	0.391	0.384	0.379	0.373	0.366	0.363	0.359	0.354	0.349	q
	q	0.500	0.497	0.494	0.492	0.491	0.490	0.489	0.489	0.488	0.488	0.487	0.486	0.486	p
0.15	u	0.333	0.352	0.373	0.383	0.389	0.393	0.396	0.399	0.403	0.406	0.408	0.411	0.414	v
	v	0.333	0.331	0.329	0.328	0.327	0.326	0.326	0.326	0.325	0.325	0.325	0.324	0.324	u
	p	0.500	0.468	0.438	0.424	0.416	0.411	0.407	0.403	0.398	0.395	0.392	0.388	0.384	q
	q	0.500	0.498	0.496	0.495	0.495	0.494	0.494	0.494	0.493	0.493	0.493	0.492	0.492	p

TABLE 10.—BEAM COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CASE III(a).—SYMMETRICAL, PARABOLIC HAUNCHES.



Values of a.		VALUES OF b.													
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
0.50	u	0.333	0.355	0.382	0.395	0.405	0.411	0.416	0.422	0.429	0.433	0.438	0.444	0.451	v
	p	0.500	0.424	0.345	0.308	0.286	0.270	0.257	0.243	0.237	0.218	0.206	0.194	0.177	q
0.40	u	0.333	0.355	0.380	0.393	0.402	0.408	0.413	0.418	0.425	0.428	0.433	0.438	0.444	v
	p	0.500	0.439	0.376	0.347	0.328	0.316	0.306	0.295	0.282	0.274	0.266	0.255	0.242	q
0.35	u	0.333	0.354	0.378	0.390	0.399	0.404	0.408	0.413	0.419	0.423	0.426	0.431	0.438	v
	p	0.500	0.447	0.392	0.366	0.350	0.338	0.330	0.320	0.309	0.302	0.295	0.286	0.274	q
0.30	u	0.333	0.353	0.375	0.386	0.393	0.398	0.403	0.407	0.412	0.415	0.419	0.423	0.428	v
	p	0.500	0.455	0.407	0.385	0.371	0.362	0.354	0.346	0.336	0.331	0.324	0.316	0.306	q
0.25	u	0.333	0.351	0.371	0.381	0.387	0.392	0.395	0.398	0.403	0.406	0.409	0.413	0.417	v
	p	0.500	0.462	0.423	0.404	0.393	0.385	0.378	0.373	0.364	0.359	0.353	0.347	0.339	q
0.20	u	0.333	0.348	0.366	0.374	0.379	0.383	0.385	0.389	0.393	0.395	0.397	0.400	0.404	v
	p	0.500	0.470	0.433	0.423	0.414	0.406	0.400	0.398	0.391	0.387	0.383	0.378	0.371	q
0.15	u	0.333	0.346	0.359	0.366	0.370	0.373	0.375	0.377	0.380	0.382	0.384	0.386	0.390	v
	p	0.500	0.477	0.454	0.442	0.436	0.431	0.427	0.423	0.418	0.415	0.412	0.408	0.403	q

TABLE 11.—BEAM COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CASE III(b).—UNSYMMETRICAL PARABOLIC HAUNCHES.



$b = \frac{I}{I'}$

Values of a .	Haunch at left support.	VALUES OF b .													Haunch at right support.
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
1.00	u	0.333	0.368	0.416	0.444	0.464	0.479	0.492	0.507	0.526	0.538	0.552	0.571	0.598	v
	v	0.333	0.311	0.283	0.267	0.257	0.249	0.242	0.236	0.225	0.219	0.212	0.204	0.192	u
	p	0.500	0.389	0.279	0.232	0.203	0.184	0.170	0.154	0.136	0.126	0.115	0.102	0.086	q
	q	0.500	0.459	0.411	0.385	0.367	0.355	0.344	0.333	0.318	0.309	0.298	0.286	0.268	p
0.50	u	0.333	0.366	0.406	0.428	0.443	0.454	0.462	0.472	0.484	0.492	0.500	0.511	0.524	v
	v	0.333	0.324	0.312	0.306	0.302	0.299	0.297	0.294	0.291	0.289	0.286	0.283	0.279	u
	p	0.500	0.434	0.367	0.337	0.318	0.306	0.296	0.285	0.273	0.266	0.257	0.247	0.235	q
	q	0.500	0.490	0.478	0.471	0.467	0.464	0.461	0.458	0.454	0.452	0.450	0.446	0.442	p
0.40	u	0.333	0.369	0.397	0.415	0.427	0.436	0.443	0.451	0.461	0.466	0.473	0.481	0.492	v
	v	0.333	0.327	0.319	0.315	0.312	0.310	0.308	0.306	0.304	0.303	0.301	0.299	0.296	u
	p	0.500	0.446	0.390	0.365	0.350	0.339	0.331	0.322	0.311	0.305	0.297	0.289	0.279	q
	q	0.500	0.494	0.486	0.482	0.479	0.477	0.475	0.473	0.471	0.469	0.468	0.466	0.463	p
0.35	u	0.333	0.360	0.391	0.408	0.418	0.426	0.432	0.439	0.447	0.452	0.458	0.465	0.474	v
	v	0.333	0.328	0.322	0.319	0.316	0.315	0.313	0.312	0.310	0.309	0.308	0.306	0.304	u
	p	0.500	0.452	0.402	0.380	0.366	0.356	0.349	0.341	0.332	0.326	0.319	0.312	0.303	q
	q	0.500	0.495	0.489	0.486	0.484	0.482	0.481	0.480	0.478	0.476	0.475	0.474	0.472	p
0.30	u	0.333	0.357	0.385	0.399	0.408	0.415	0.420	0.426	0.433	0.437	0.442	0.448	0.455	v
	v	0.333	0.329	0.325	0.322	0.320	0.319	0.318	0.317	0.316	0.315	0.314	0.313	0.311	u
	p	0.500	0.458	0.415	0.395	0.383	0.375	0.368	0.361	0.353	0.348	0.342	0.336	0.327	q
	q	0.500	0.496	0.492	0.490	0.488	0.487	0.486	0.485	0.484	0.483	0.482	0.480	0.479	p
0.25	u	0.333	0.354	0.378	0.390	0.397	0.403	0.407	0.412	0.418	0.421	0.425	0.430	0.436	v
	v	0.333	0.330	0.327	0.325	0.324	0.323	0.322	0.322	0.321	0.320	0.319	0.318	0.317	u
	p	0.500	0.464	0.428	0.411	0.401	0.394	0.388	0.382	0.375	0.371	0.366	0.360	0.353	q
	q	0.500	0.497	0.494	0.493	0.492	0.491	0.490	0.490	0.489	0.488	0.487	0.487	0.486	p
0.20	u	0.333	0.351	0.370	0.380	0.386	0.390	0.394	0.398	0.402	0.405	0.408	0.412	0.417	v
	v	0.333	0.331	0.329	0.328	0.327	0.327	0.326	0.326	0.325	0.325	0.324	0.324	0.323	u
	p	0.500	0.471	0.441	0.428	0.420	0.414	0.409	0.404	0.398	0.395	0.391	0.386	0.380	q
	q	0.500	0.498	0.496	0.496	0.495	0.494	0.494	0.493	0.493	0.492	0.492	0.491	0.491	p
0.15	u	0.333	0.347	0.362	0.369	0.374	0.377	0.379	0.382	0.386	0.388	0.390	0.393	0.396	v
	v	0.333	0.332	0.331	0.330	0.330	0.330	0.329	0.329	0.328	0.328	0.328	0.327	0.327	u
	p	0.500	0.478	0.456	0.445	0.439	0.434	0.431	0.427	0.422	0.420	0.416	0.413	0.408	q
	q	0.500	0.499	0.498	0.497	0.497	0.497	0.496	0.496	0.496	0.496	0.496	0.496	0.495	p

TABLE 12.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.

CONCENTRATED UNIT LOAD.

CASE I(a).—SYMMETRICAL, SHARPLY CURVED HAUNCHES.

VALUES OF s , READ DOWN.

VALUES OF:

Concentrated unit load at Point:

a	b	1	2	3	4	5	6	7	8	9	10	11
....	1.00	0.0488	0.0849	0.1094	0.1235	0.1283	0.1250	0.1148	0.0988	0.0781	0.0540	0.0276
0.50	0.20	0.0482	0.0894	0.1317	0.1495	0.1540	0.1528	0.1408	0.1202	0.0987	0.0638	0.0328
	0.10	0.0478	0.0904	0.1241	0.1478	0.1598	0.1590	0.1466	0.1249	0.0972	0.0663	0.0333
	0.05	0.0478	0.0911	0.1258	0.1504	0.1630	0.1626	0.1495	0.1275	0.0993	0.0674	0.0339
	0.03	0.0478	0.0914	0.1265	0.1516	0.1643	0.1643	0.1512	0.1289	0.1001	0.0680	0.0342
0.40	0.20	0.0480	0.0901	0.1224	0.1443	0.1540	0.1525	0.1408	0.1201	0.0940	0.0643	0.0324
	0.10	0.0481	0.0910	0.1247	0.1484	0.1592	0.1580	0.1466	0.1243	0.0970	0.0662	0.0333
	0.05	0.0480	0.0915	0.1261	0.1505	0.1619	0.1607	0.1485	0.1265	0.0989	0.0671	0.0338
	0.03	0.0479	0.0918	0.1270	0.1516	0.1630	0.1622	0.1495	0.1278	0.0995	0.0676	0.0340
0.35	0.20	0.0485	0.0904	0.1226	0.1439	0.1529	0.1509	0.1398	0.1195	0.0935	0.0640	0.0322
	0.10	0.0486	0.0912	0.1250	0.1477	0.1577	0.1557	0.1438	0.1230	0.0964	0.0657	0.0331
	0.05	0.0484	0.0920	0.1263	0.1497	0.1598	0.1582	0.1461	0.1251	0.0981	0.0666	0.0335
	0.03	0.0483	0.0925	0.1270	0.1505	0.1611	0.1594	0.1473	0.1262	0.0986	0.0669	0.0337
0.30	0.20	0.0489	0.0909	0.1228	0.1433	0.1513	0.1486	0.1371	0.1178	0.0928	0.0632	0.0320
	0.10	0.0488	0.0917	0.1252	0.1464	0.1551	0.1527	0.1409	0.1212	0.0949	0.0649	0.0327
	0.05	0.0486	0.0925	0.1264	0.1483	0.1570	0.1551	0.1431	0.1228	0.0965	0.0658	0.0331
	0.03	0.0488	0.0925	0.1269	0.1489	0.1582	0.1568	0.1437	0.1237	0.0969	0.0661	0.0332
0.25	0.20	0.0491	0.0913	0.1225	0.1415	0.1487	0.1460	0.1346	0.1156	0.0913	0.0624	0.0316
	0.10	0.0493	0.0923	0.1245	0.1443	0.1516	0.1493	0.1377	0.1183	0.0935	0.0640	0.0322
	0.05	0.0498	0.0928	0.1257	0.1457	0.1535	0.1508	0.1390	0.1198	0.0944	0.0646	0.0325
	0.03	0.0492	0.0929	0.1259	0.1464	0.1543	0.1519	0.1401	0.1204	0.0948	0.0649	0.0327
a	b	11	10	9	8	7	6	5	4	3	2	1

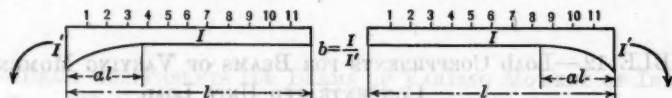
Concentrated unit load at Point:

VALUES OF:

VALUES OF t , READ UP.

TABLE 13.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CONCENTRATED UNIT LOAD.

CASE I(b).—UNSYMMETRICAL, SHARPLY CURVED HAUNCH.



VALUES OF:		Haunch at left support.	CONCENTRATED UNIT LOAD AT POINT:											Haunch at right support.
a.	b.		1	2	3	4	5	6	7	8	9	10	11	
....	1.00	s	0.0488	0.0640	0.1094	0.1235	0.1288	0.1250	0.1148	0.0988	0.0781	0.0540	0.0276	t
		t	0.0276	0.0540	0.0781	0.0988	0.1148	0.1250	0.1288	0.1235	0.1094	0.0849	0.0488	s
	0.20	s	0.0441	0.0819	0.1106	0.1307	0.1414	0.1427	0.1359	0.1200	0.0973	0.0681	0.0356	t
		t	0.0226	0.0450	0.0660	0.0854	0.1017	0.1137	0.1200	0.1181	0.1088	0.0870	0.0515	s
1.00	0.10	s	0.0431	0.0807	0.1109	0.1329	0.1453	0.1485	0.1421	0.1265	0.1081	0.0729	0.0380	t
		t	0.0217	0.0432	0.0639	0.0828	0.0993	0.1116	0.1184	0.1181	0.1085	0.0872	0.0521	s
	0.05	s	0.0420	0.0797	0.1111	0.1340	0.1478	0.1511	0.1459	0.1306	0.1068	0.0758	0.0396	t
		t	0.0212	0.0425	0.0627	0.0817	0.0980	0.1108	0.1178	0.1178	0.1085	0.0874	0.0525	s
	0.03	s	0.0417	0.0795	0.1112	0.1345	0.1490	0.1528	0.1480	0.1325	0.1082	0.0767	0.0403	t
		t	0.0210	0.0421	0.0624	0.0812	0.0974	0.1104	0.1174	0.1177	0.1085	0.0875	0.0525	s
	0.20	s	0.0455	0.0846	0.1156	0.1373	0.1488	0.1500	0.1411	0.1232	0.0986	0.0684	0.0353	t
		t	0.0252	0.0501	0.0738	0.0950	0.1126	0.1251	0.1299	0.1263	0.1126	0.0890	0.0507	s
0.50	0.10	s	0.0448	0.0847	0.1170	0.1401	0.1529	0.1551	0.1463	0.1281	0.1024	0.0714	0.0368	t
		t	0.0249	0.0495	0.0731	0.0944	0.1124	0.1250	0.1309	0.1268	0.1133	0.0884	0.0512	s
	0.05	s	0.0445	0.0848	0.1175	0.1420	0.1553	0.1579	0.1492	0.1306	0.1048	0.0730	0.0376	t
		t	0.0247	0.0494	0.0729	0.0943	0.1123	0.1250	0.1308	0.1271	0.1134	0.0887	0.0512	s
	0.03	s	0.0444	0.0846	0.1180	0.1423	0.1563	0.1588	0.1501	0.1318	0.1057	0.0737	0.0380	t
		t	0.0246	0.0492	0.0727	0.0942	0.1122	0.1250	0.1309	0.1271	0.1136	0.0887	0.0513	s
	0.20	s	0.0464	0.0863	0.1180	0.1396	0.1499	0.1495	0.1392	0.1209	0.0965	0.0671	0.0344	t
		t	0.0250	0.0516	0.0757	0.0972	0.1148	0.1263	0.1304	0.1260	0.1126	0.0872	0.0503	s
0.40	0.10	s	0.0459	0.0869	0.1196	0.1423	0.1539	0.1539	0.1436	0.1252	0.0994	0.0694	0.0356	t
		t	0.0257	0.0513	0.0753	0.0970	0.1146	0.1264	0.1304	0.1263	0.1126	0.0874	0.0506	s
	0.05	s	0.0458	0.0871	0.1204	0.1440	0.1568	0.1562	0.1458	0.1270	0.1012	0.0707	0.0362	t
		t	0.0256	0.0511	0.0752	0.0968	0.1147	0.1264	0.1307	0.1264	0.1126	0.0876	0.0506	s
	0.03	s	0.0457	0.0871	0.1209	0.1447	0.1570	0.1570	0.1469	0.1280	0.1020	0.0708	0.0365	t
		t	0.0256	0.0508	0.0752	0.0970	0.1149	0.1265	0.1309	0.1266	0.1127	0.0878	0.0506	s
	0.20	s	0.0468	0.0875	0.1193	0.1401	0.1494	0.1482	0.1376	0.1193	0.0940	0.0660	0.0338	t
		t	0.0263	0.0523	0.0767	0.0982	0.1153	0.1264	0.1302	0.1258	0.1115	0.0868	0.0501	s
	0.10	s	0.0466	0.0882	0.1209	0.1432	0.1533	0.1524	0.1415	0.1230	0.0978	0.0680	0.0349	t
		t	0.0261	0.0519	0.0764	0.0982	0.1154	0.1265	0.1302	0.1258	0.1119	0.0871	0.0501	s
0.35	0.05	s	0.0465	0.0889	0.1221	0.1447	0.1554	0.1545	0.1438	0.1246	0.0994	0.0691	0.0354	t
		t	0.0260	0.0517	0.0764	0.0981	0.1154	0.1265	0.1304	0.1260	0.1119	0.0872	0.0502	s
	0.03	s	0.0464	0.0885	0.1224	0.1452	0.1559	0.1550	0.1447	0.1254	0.1001	0.0696	0.0356	t
		t	0.0260	0.0516	0.0763	0.0980	0.1155	0.1265	0.1305	0.1261	0.1120	0.0873	0.0502	s
	0.20	s	0.0479	0.0886	0.1201	0.1401	0.1485	0.1465	0.1357	0.1173	0.0930	0.0647	0.0331	t
		t	0.0266	0.0529	0.0773	0.0988	0.1154	0.1263	0.1304	0.1256	0.1119	0.0862	0.0497	s
0.30	0.10	s	0.0476	0.0894	0.1218	0.1431	0.1517	0.1501	0.1390	0.1205	0.0955	0.0664	0.0340	t
		t	0.0265	0.0526	0.0772	0.0989	0.1154	0.1263	0.1301	0.1254	0.1113	0.0864	0.0497	s
	0.05	s	0.0474	0.0897	0.1230	0.1442	0.1533	0.1517	0.1409	0.1217	0.0968	0.0673	0.0345	t
		t	0.0264	0.0525	0.0772	0.0989	0.1154	0.1264	0.1301	0.1256	0.1114	0.0865	0.0499	s
	0.03	s	0.0473	0.0899	0.1234	0.1451	0.1543	0.1526	0.1418	0.1225	0.0974	0.0677	0.0347	t
		t	0.0264	0.0525	0.0772	0.0989	0.1154	0.1265	0.1304	0.1256	0.1115	0.0868	0.0499	s

TABLE 13.—(Continued.)

VALUES OF:		Haunch at left support.	CONCENTRATED UNIT LOAD AT POINT:										Haunch at right support.	
α .	b .		1	2	3	4	5	6	7	8	9	10		11
0.25	0.20	s	0.0484	0.0897	0.1206	0.1393	0.1467	0.1440	0.1381	0.1151	0.0918	0.0632	0.0324	t
		t	0.0269	0.0532	0.0778	0.0992	0.1154	0.1261	0.1296	0.1248	0.1108	0.0859	0.0495	s
	0.10	s	0.0483	0.0907	0.1222	0.1418	0.1498	0.1470	0.1362	0.1174	0.0935	0.0647	0.0332	t
		t	0.0268	0.0533	0.0778	0.0992	0.1156	0.1263	0.1299	0.1250	0.1108	0.0860	0.0495	s
	0.05	s	0.0483	0.0909	0.1233	0.1431	0.1511	0.1487	0.1374	0.1188	0.0946	0.0655	0.0335	t
		t	0.0268	0.0532	0.0778	0.0992	0.1157	0.1264	0.1299	0.1251	0.1109	0.0861	0.0495	s
	0.03	s	0.0483	0.0909	0.1238	0.1437	0.1518	0.1494	0.1380	0.1193	0.0950	0.0658	0.0337	t
		t	0.0267	0.0532	0.0778	0.0992	0.1157	0.1264	0.1299	0.1253	0.1111	0.0861	0.0496	s

TABLE 14.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CONCENTRATED UNIT LOAD.

CASE II(a).—SYMMETRICAL, STRAIGHT HAUNCHES.



VALUES OF:		VALUES OF δ , READ DOWN.										
		Concentrated unit load at Point.										
α .	δ .	1	2	3	4	5	6	7	8	9	10	11
....	1.00	0.0488	0.0849	0.1094	0.1285	0.1288	0.1250	0.1148	0.0988	0.0781	0.0540	0.0276
0.50	0.20	0.0478	0.0879	0.1206	0.1448	0.1573	0.1577	0.1447	0.1226	0.0954	0.0650	0.0327
	0.10	0.0466	0.0881	0.1235	0.1509	0.1679	0.1709	0.1563	0.1318	0.1014	0.0686	0.0345
	0.05	0.0456	0.0876	0.1250	0.1558	0.1770	0.1827	0.1666	0.1393	0.1067	0.0720	0.0360
	0.03	0.0450	0.0874	0.1262	0.1592	0.1838	0.1908	0.1739	0.1440	0.1101	0.0740	0.0370
0.40	0.20	0.0471	0.0885	0.1227	0.1476	0.1618	0.1608	0.1484	0.1259	0.0977	0.0664	0.0334
	0.10	0.0463	0.0888	0.1254	0.1552	0.1726	0.1739	0.1607	0.1356	0.1041	0.0706	0.0353
	0.05	0.0453	0.0884	0.1274	0.1607	0.1825	0.1856	0.1714	0.1434	0.1097	0.0737	0.0369
	0.03	0.0445	0.0877	0.1292	0.1636	0.1882	0.1928	0.1777	0.1482	0.1127	0.0754	0.0377
0.35	0.20	0.0474	0.0892	0.1234	0.1483	0.1604	0.1596	0.1471	0.1258	0.0977	0.0663	0.0334
	0.10	0.0466	0.0894	0.1271	0.1560	0.1712	0.1712	0.1585	0.1348	0.1038	0.0704	0.0352
	0.05	0.0458	0.0890	0.1291	0.1617	0.1798	0.1811	0.1674	0.1419	0.1088	0.0731	0.0366
	0.03	0.0450	0.0887	0.1299	0.1644	0.1851	0.1869	0.1729	0.1463	0.1117	0.0747	0.0374
0.30	0.20	0.0475	0.0899	0.1246	0.1482	0.1586	0.1570	0.1450	0.1242	0.0971	0.0659	0.0332
	0.10	0.0470	0.0906	0.1283	0.1556	0.1682	0.1669	0.1548	0.1325	0.1028	0.0696	0.0348
	0.05	0.0465	0.0908	0.1308	0.1612	0.1755	0.1751	0.1620	0.1386	0.1073	0.0721	0.0361
	0.03	0.0461	0.0904	0.1316	0.1644	0.1799	0.1804	0.1671	0.1427	0.1099	0.0736	0.0368
0.25	0.20	0.0485	0.0911	0.1254	0.1467	0.1554	0.1530	0.1412	0.1215	0.0954	0.0651	0.0327
	0.10	0.0478	0.0923	0.1293	0.1582	0.1635	0.1619	0.1495	0.1235	0.1005	0.0684	0.0342
	0.05	0.0472	0.0923	0.1321	0.1585	0.1700	0.1688	0.1565	0.1323	0.1045	0.0705	0.0353
	0.03	0.0471	0.0924	0.1335	0.1611	0.1732	0.1719	0.1594	0.1369	0.1067	0.0717	0.0358
α	δ .	11	10	9	8	7	6	5	4	3	2	1
		Concentrated unit load at Point.										
VALUES OF:		VALUES OF t , READ UP.										

TABLE 15.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CONCENTRATED UNIT LOAD.

CASE II(b).—UNSYMMETRICAL, STRAIGHT HAUNCH.

VALUES OF :

CONCENTRATED UNIT LOAD AT POINT.

α	b	Haunch at left support.	1	2	3	4	5	6	7	8	9	10	11	Haunch at right support.	
1.00	1.00	s	0.0488	0.0849	0.1094	0.1235	0.1283	0.1250	0.1148	0.0988	0.0781	0.0540	0.0276	t	
		t	0.0276	0.0540	0.0781	0.0988	0.1148	0.1250	0.1283	0.1235	0.1094	0.0849	0.0488	s	
	0.20	s	0.0440	0.0790	0.1063	0.1247	0.1349	0.1370	0.1306	0.1165	0.0952	0.0675	0.0350	t	
		t	0.0350	0.0675	0.0952	0.1063	0.1247	0.1349	0.1370	0.1306	0.1165	0.0952	0.0675	s	
	0.10	s	0.0412	0.0754	0.1026	0.1222	0.1339	0.1377	0.1335	0.1213	0.1007	0.0726	0.0384	t	
		t	0.0384	0.0726	0.1007	0.1222	0.1339	0.1377	0.1335	0.1213	0.1007	0.0726	0.0384	s	
	0.05	s	0.0382	0.0709	0.0979	0.1178	0.1310	0.1371	0.1351	0.1244	0.1050	0.0775	0.0413	t	
		t	0.0413	0.0775	0.1050	0.1244	0.1351	0.1371	0.1351	0.1244	0.1050	0.0775	0.0413	s	
	0.03	s	0.0361	0.0674	0.0933	0.1138	0.1273	0.1354	0.1349	0.1257	0.1079	0.0804	0.0432	t	
		t	0.0432	0.0804	0.1079	0.1257	0.1349	0.1354	0.1273	0.1257	0.1079	0.0804	0.0432	s	
	0.50	0.20	s	0.0428	0.0800	0.1110	0.1346	0.1496	0.1542	0.1469	0.1301	0.1046	0.0732	0.0377	t
			t	0.0377	0.0732	0.1046	0.1301	0.1469	0.1542	0.1496	0.1346	0.1110	0.0800	0.0428	s
0.10		s	0.0401	0.0766	0.1087	0.1347	0.1537	0.1625	0.1577	0.1407	0.1142	0.0801	0.0416	t	
		t	0.0416	0.0801	0.1142	0.1407	0.1577	0.1625	0.1537	0.1347	0.1087	0.0766	0.0401	s	
0.05		s	0.0372	0.0729	0.1053	0.1333	0.1555	0.1684	0.1663	0.1501	0.1225	0.0864	0.0443	t	
		t	0.0443	0.0864	0.1225	0.1501	0.1663	0.1684	0.1555	0.1333	0.1053	0.0729	0.0372	s	
0.03		s	0.0355	0.0699	0.1020	0.1314	0.1556	0.1714	0.1708	0.1550	0.1274	0.0896	0.0462	t	
		t	0.0462	0.0896	0.1274	0.1550	0.1708	0.1714	0.1556	0.1314	0.1020	0.0699	0.0355	s	
0.20		s	0.0441	0.0830	0.1152	0.1399	0.1536	0.1554	0.1461	0.1280	0.1024	0.0711	0.0366	t	
		t	0.0366	0.0711	0.1024	0.1280	0.1461	0.1554	0.1536	0.1399	0.1152	0.0830	0.0441	s	
0.10		s	0.0421	0.0809	0.1149	0.1432	0.1609	0.1647	0.1561	0.1374	0.1108	0.0771	0.0397	t	
		t	0.0397	0.0771	0.1108	0.1374	0.1561	0.1647	0.1609	0.1432	0.1149	0.0809	0.0421	s	
0.40	0.05	s	0.0402	0.0784	0.1135	0.1440	0.1654	0.1715	0.1644	0.1451	0.1171	0.0819	0.0422	t	
		t	0.0422	0.0819	0.1171	0.1451	0.1644	0.1715	0.1654	0.1440	0.1135	0.0784	0.0402	s	
	0.03	s	0.0390	0.0769	0.1122	0.1438	0.1675	0.1754	0.1686	0.1496	0.1206	0.0848	0.0487	t	
		t	0.0487	0.0848	0.1206	0.1496	0.1686	0.1754	0.1675	0.1438	0.1122	0.0769	0.0390	s	
	0.20	s	0.0450	0.0848	0.1181	0.1419	0.1545	0.1545	0.1445	0.1259	0.1004	0.0697	0.0359	t	
		t	0.0359	0.0697	0.1004	0.1259	0.1445	0.1545	0.1545	0.1419	0.1181	0.0848	0.0450	s	
	0.10	s	0.0433	0.0833	0.1186	0.1465	0.1619	0.1688	0.1540	0.1344	0.1079	0.0749	0.0386	t	
		t	0.0386	0.0749	0.1079	0.1344	0.1540	0.1688	0.1619	0.1465	0.1186	0.0833	0.0433	s	
	0.05	s	0.0417	0.0815	0.1183	0.1487	0.1668	0.1703	0.1609	0.1413	0.1134	0.0790	0.0407	t	
		t	0.0407	0.0790	0.1134	0.1413	0.1609	0.1703	0.1668	0.1487	0.1183	0.0815	0.0417	s	
	0.03	s	0.0409	0.0804	0.1173	0.1496	0.1693	0.1739	0.1648	0.1450	0.1167	0.0814	0.0419	t	
		t	0.0419	0.0814	0.1167	0.1450	0.1648	0.1739	0.1693	0.1496	0.1173	0.0804	0.0409	s	
0.30	0.20	s	0.0460	0.0866	0.1203	0.1435	0.1537	0.1528	0.1423	0.1233	0.0984	0.0684	0.0350	t	
		t	0.0350	0.0684	0.0984	0.1233	0.1423	0.1528	0.1537	0.1435	0.1203	0.0866	0.0460	s	
	0.10	s	0.0445	0.0859	0.1222	0.1485	0.1611	0.1611	0.1507	0.1309	0.1044	0.0729	0.0373	t	
		t	0.0373	0.0729	0.1044	0.1309	0.1507	0.1611	0.1611	0.1485	0.1222	0.0859	0.0445	s	
	0.05	s	0.0433	0.0847	0.1226	0.1516	0.1657	0.1671	0.1567	0.1365	0.1092	0.0758	0.0391	t	
		t	0.0391	0.0758	0.1092	0.1365	0.1567	0.1671	0.1657	0.1516	0.1226	0.0847	0.0433	s	
	0.03	s	0.0425	0.0841	0.1227	0.1532	0.1686	0.1701	0.1599	0.1401	0.1121	0.0778	0.0401	t	
		t	0.0401	0.0778	0.1121	0.1401	0.1599	0.1701	0.1686	0.1532	0.1227	0.0841	0.0425	s	

ers.
TIA.

TABLE 15.—(Continued.)

VALUES OF :		Haunch at left support.	CONCENTRATED UNIT LOAD AT POINT :											Haunch at right support.
<i>a</i>	<i>b</i>		1	2	3	4	5	6	7	8	9	10	11	
0.25	0.20	<i>s</i>	0.0472	0.0887	0.1228	0.1433	0.1523	0.1502	0.1392	0.1207	0.0956	0.0665	0.0341	<i>t</i>
		<i>t</i>	0.0265	0.0527	0.0776	0.0990	0.1159	0.1264	0.1302	0.1254	0.1114	0.0865	0.0498	<i>s</i>
	0.10	<i>s</i>	0.0464	0.0888	0.1248	0.1496	0.1586	0.1573	0.1464	0.1269	0.1009	0.0702	0.0360	<i>t</i>
		<i>t</i>	0.0262	0.0523	0.0772	0.0990	0.1160	0.1270	0.1308	0.1261	0.1119	0.0870	0.0501	<i>s</i>
	0.05	<i>s</i>	0.0450	0.0882	0.1265	0.1517	0.1630	0.1625	0.1513	0.1315	0.1049	0.0729	0.0374	<i>t</i>
		<i>t</i>	0.0259	0.0518	0.0768	0.0989	0.1161	0.1273	0.1310	0.1267	0.1123	0.0874	0.0502	<i>s</i>
	0.03	<i>s</i>	0.0446	0.0879	0.1270	0.1537	0.1657	0.1657	0.1546	0.1344	0.1072	0.0746	0.0382	<i>t</i>
		<i>t</i>	0.0257	0.0515	0.0766	0.0989	0.1163	0.1275	0.1315	0.1268	0.1126	0.0878	0.0505	<i>s</i>

TABLE 16.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CONCENTRATED UNIT LOAD.

CASE III(a).—SYMMETRICAL, PARABOLIC HAUNCHES.

VALUES OF:		VALUES OF <i>s</i> . READ DOWN.										
<i>a</i>	<i>b</i>	CONCENTRATED UNIT LOAD AT POINT:										
1.00	1.00	0.0488	0.0849	0.1094	0.1235	0.1283	0.1250	0.1148	0.0988	0.0781	0.0540	0.0276
0.50	0.20	0.0478	0.0893	0.1229	0.1459	0.1509	0.1561	0.1439	0.1225	0.0957	0.0652	0.0328
	0.10	0.0474	0.0898	0.1260	0.1526	0.1672	0.1676	0.1543	0.1310	0.1015	0.0690	0.0345
	0.05	0.0464	0.0897	0.1288	0.1586	0.1760	0.1777	0.1688	0.1393	0.1066	0.0719	0.0360
	0.03	0.0457	0.0892	0.1291	0.1615	0.1814	0.1841	0.1695	0.1428	0.1096	0.0736	0.0368
0.40	0.20	0.0480	0.0905	0.1235	0.1459	0.1554	0.1538	0.1420	0.1215	0.0952	0.0649	0.0326
	0.10	0.0474	0.0908	0.1275	0.1536	0.1650	0.1684	0.1514	0.1295	0.1002	0.0685	0.0342
	0.05	0.0467	0.0908	0.1293	0.1580	0.1726	0.1721	0.1593	0.1357	0.1053	0.0711	0.0355
	0.03	0.0464	0.0906	0.1305	0.1616	0.1773	0.1778	0.1647	0.1401	0.1082	0.0727	0.0363
0.35	0.20	0.0484	0.0906	0.1238	0.1448	0.1538	0.1515	0.1398	0.1199	0.0941	0.0644	0.0324
	0.10	0.0480	0.0915	0.1276	0.1515	0.1621	0.1605	0.1482	0.1270	0.0992	0.0674	0.0339
	0.05	0.0473	0.0917	0.1302	0.1568	0.1690	0.1682	0.1555	0.1331	0.1035	0.0702	0.0351
	0.03	0.0470	0.0919	0.1311	0.1600	0.1734	0.1730	0.1600	0.1368	0.1061	0.0716	0.0358
0.30	0.20	0.0486	0.0911	0.1226	0.1436	0.1514	0.1490	0.1371	0.1181	0.0931	0.0637	0.0320
	0.10	0.0483	0.0922	0.1273	0.1498	0.1590	0.1566	0.1445	0.1244	0.0978	0.0664	0.0334
	0.05	0.0480	0.0926	0.1304	0.1545	0.1649	0.1632	0.1508	0.1296	0.1013	0.0689	0.0345
	0.03	0.0479	0.0931	0.1321	0.1575	0.1689	0.1672	0.1545	0.1329	0.1037	0.0703	0.0351
0.25	0.20	0.0495	0.0917	0.1229	0.1416	0.1484	0.1458	0.1344	0.1157	0.0913	0.0624	0.0316
	0.10	0.0490	0.0932	0.1268	0.1469	0.1548	0.1525	0.1406	0.1209	0.0952	0.0652	0.0328
	0.05	0.0487	0.0938	0.1295	0.1515	0.1600	0.1576	0.1454	0.1255	0.0987	0.0674	0.0337
	0.03	0.0483	0.0941	0.1313	0.1540	0.1631	0.1610	0.1486	0.1280	0.1007	0.0683	0.0343
<i>a</i>	<i>b</i>	11	10	9	8	7	6	5	4	3	2	1
VALUES OF:		CONCENTRATED UNIT LOAD AT POINT:										
		VALUES OF <i>t</i> . READ UP.										

support.

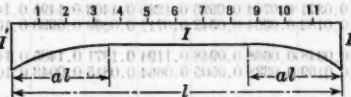
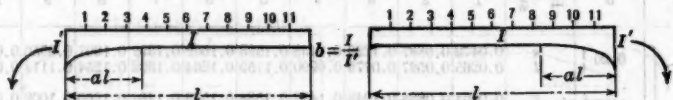


TABLE 17.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
CONCENTRATED UNIT LOAD.

CASE III(b).—UNSYMMETRICAL, PARABOLIC HAUNCH.



VALUES OF:		Haunch at left support.	CONCENTRATED UNIT LOAD AT POINT:											Haunch at right support.
α .	b .		1	2	3	4	5	6	7	8	9	10	11	
1.00	1.00	s	0.0488	0.0849	0.1094	0.1235	0.1283	0.1250	0.1148	0.0988	0.0781	0.0540	0.0276	t
		t	0.0276	0.0540	0.0781	0.0988	0.1148	0.1250	0.1283	0.1235	0.1094	0.0849	0.0488	t
	0.90	s	0.0435	0.0798	0.1087	0.1295	0.1415	0.1442	0.1384	0.1283	0.1003	0.0705	0.0364	t
		t	0.0222	0.0441	0.0649	0.0842	0.1007	0.1183	0.1202	0.1200	0.1101	0.0882	0.0524	s
	0.10	s	0.0408	0.0757	0.1047	0.1274	0.1421	0.1480	0.1446	0.1313	0.1087	0.0777	0.0403	t
		t	0.0201	0.0402	0.0596	0.0778	0.0938	0.1071	0.1154	0.1168	0.1088	0.0887	0.0533	s
	0.05	s	0.0371	0.0704	0.0995	0.1286	0.1408	0.1495	0.1484	0.1376	0.1156	0.0889	0.0441	t
		t	0.0254	0.0505	0.0744	0.0911	0.0989	0.0998	0.1038	0.1123	0.1066	0.0886	0.0542	s
	0.03	s	0.0348	0.0668	0.0948	0.1194	0.1371	0.1485	0.1497	0.1405	0.1200	0.0880	0.0468	t
		t	0.0169	0.0338	0.0505	0.0664	0.0815	0.0943	0.1043	0.1088	0.1045	0.0880	0.0546	s
	0.20	s	0.0454	0.0852	0.1174	0.1401	0.1516	0.1525	0.1436	0.1248	0.0994	0.0680	0.0355	t
		t	0.0254	0.0505	0.0744	0.0961	0.1139	0.1258	0.1304	0.1265	0.1127	0.0879	0.0505	s
0.50	0.10	s	0.0434	0.0832	0.1173	0.1428	0.1580	0.1608	0.1520	0.1331	0.1067	0.0744	0.0383	t
		t	0.0246	0.0498	0.0730	0.0946	0.1130	0.1258	0.1308	0.1273	0.1136	0.0887	0.0513	s
	0.05	s	0.0418	0.0806	0.1160	0.1445	0.1622	0.1671	0.1592	0.1406	0.1130	0.0791	0.0408	t
		t	0.0240	0.0479	0.0713	0.0932	0.1123	0.1255	0.1312	0.1281	0.1146	0.0886	0.0519	s
	0.03	s	0.0404	0.0792	0.1144	0.1440	0.1645	0.1711	0.1640	0.1451	0.1170	0.0822	0.0424	t
		t	0.0235	0.0470	0.0702	0.0923	0.1115	0.1254	0.1313	0.1285	0.1152	0.0903	0.0524	s
	0.20	s	0.0465	0.0871	0.1195	0.1415	0.1514	0.1506	0.1398	0.1216	0.0967	0.0672	0.0344	t
		t	0.0261	0.0519	0.0761	0.0978	0.1151	0.1265	0.1305	0.1260	0.1119	0.0871	0.0500	s
	0.10	s	0.0452	0.0863	0.1213	0.1461	0.1581	0.1585	0.1479	0.1289	0.1027	0.0717	0.0368	t
		t	0.0256	0.0512	0.0752	0.0974	0.1153	0.1267	0.1310	0.0967	0.1128	0.0879	0.0506	s
	0.05	s	0.0438	0.0853	0.1212	0.1487	0.1631	0.1645	0.1547	0.1352	0.1081	0.0750	0.0387	t
		t	0.0251	0.0502	0.0744	0.0968	0.1150	0.1268	0.1313	0.1274	0.1135	0.0884	0.0508	s
0.40	0.03	s	0.0428	0.0849	0.1208	0.1502	0.1665	0.1685	0.1588	0.1391	0.1112	0.0775	0.0399	t
		t	0.0248	0.0496	0.0738	0.0965	0.1147	0.1269	0.1317	0.1278	0.1138	0.0887	0.0511	s
	0.20	s	0.0473	0.0884	0.1206	0.1414	0.1508	0.1488	0.1382	0.1198	0.0950	0.0660	0.0388	t
		t	0.0265	0.0526	0.0771	0.0985	0.1155	0.1265	0.1308	0.1256	0.1115	0.0867	0.0497	s
	0.10	s	0.0462	0.0881	0.1226	0.1463	0.1571	0.1563	0.1455	0.1261	0.1006	0.0699	0.0358	t
		t	0.0260	0.0520	0.0764	0.0983	0.1156	0.1265	0.1306	0.1252	0.1121	0.0872	0.0501	s
	0.05	s	0.0448	0.0873	0.1239	0.1496	0.1623	0.1623	0.1515	0.1316	0.1052	0.0732	0.0375	t
		t	0.0256	0.0513	0.0760	0.0980	0.1155	0.1269	0.1309	0.1266	0.1127	0.0878	0.0508	s
	0.03	s	0.0441	0.0864	0.1241	0.1515	0.1650	0.1654	0.1552	0.1352	0.1078	0.0748	0.0386	t
		t	0.0254	0.0506	0.0756	0.0979	0.1157	0.1270	0.1313	0.1270	0.1133	0.0882	0.0508	s
	0.20	s	0.0479	0.0898	0.1212	0.1412	0.1499	0.1468	0.1356	0.1172	0.0933	0.0646	0.0331	t
		t	0.0267	0.0531	0.0776	0.0989	0.1156	0.1263	0.1300	0.1253	0.1111	0.0863	0.0496	s
0.30	0.10	s	0.0470	0.0899	0.1239	0.1457	0.1549	0.1533	0.1423	0.1230	0.0978	0.0680	0.0349	t
		t	0.0264	0.0525	0.0773	0.0990	0.1158	0.1266	0.1305	0.1257	0.1117	0.0866	0.0500	s
	0.05	s	0.0463	0.0896	0.1255	0.1495	0.1600	0.1587	0.1473	0.1281	0.1019	0.0708	0.0363	t
		t	0.0261	0.0522	0.0771	0.0968	0.1158	0.1268	0.1306	0.1262	0.1120	0.0872	0.0501	s
	0.03	s	0.0452	0.0891	0.1262	0.1513	0.1625	0.1616	0.1508	0.1307	0.1043	0.0725	0.0372	t
		t	0.0259	0.0519	0.0769	0.0968	0.1159	0.1272	0.1309	0.1266	0.1122	0.0873	0.0503	s

TABLE 17.—(Continued.)

VALUES OF:		Haunch at left support.	CONCENTRATED UNIT LOAD AT POINT.										Haunch at right support.	
a.	b.		1	2	3	4	5	6	7	8	9	10		11
0.25	0.20	s	0.0487	0.0904	0.1212	0.1399	0.1470	0.1442	0.1333	0.1150	0.0912	0.0631	0.0324	t
		t	0.0270	0.0537	0.0780	0.0992	0.1154	0.1262	0.1294	0.1249	0.1106	0.0859	0.0494	s
	0.10	s	0.0480	0.0912	0.1242	0.1441	0.1523	0.1498	0.1388	0.1197	0.0953	0.0660	0.0328	t
		t	0.0268	0.0532	0.0780	0.0993	0.1158	0.1264	0.1300	0.1251	0.1109	0.0861	0.0497	s
	0.05	s	0.0476	0.0914	0.1263	0.1473	0.1566	0.1545	0.1432	0.1238	0.0981	0.0682	0.0350	t
		t	0.0266	0.0531	0.0778	0.0993	0.1158	0.1267	0.1302	0.1254	0.1113	0.0864	0.0499	s
	0.03	s	0.0469	0.0916	0.1273	0.1497	0.1592	0.1573	0.1458	0.1265	0.1002	0.0692	0.0357	t
		t	0.0264	0.0529	0.0777	0.0994	0.1159	0.1268	0.1306	0.1258	0.1115	0.0866	0.0500	s

TABLE 18.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
TOTAL LOAD 1, UNIFORMLY DISTRIBUTED.
CASE I(a).—SYMMETRICAL, SHARPLY CURVED HAUNCHES.



Values of a:		VALUES OF b:													
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
0.50	s	0.0833	0.0885	0.0946	0.0972	0.0987	0.0997	0.1004	0.1011	0.1018	0.1022	0.1026	0.1030	0.1033	t
0.40	s	0.0833	0.0891	0.0950	0.0975	0.0987	0.0996	0.1002	0.1008	0.1014	0.1017	0.1020	0.1024	0.1027	t
0.35	s	0.0833	0.0891	0.0948	0.0970	0.0982	0.0990	0.0996	0.1000	0.1006	0.1009	0.1012	0.1014	0.1018	t
0.30	s	0.0833	0.0889	0.0942	0.0963	0.0973	0.0980	0.0984	0.0989	0.0994	0.0996	0.0999	0.1001	0.1003	t
0.25	s	0.0833	0.0885	0.0932	0.0951	0.0960	0.0966	0.0970	0.0973	0.0977	0.0979	0.0981	0.0984	0.0986	t
0.20	s	0.0833	0.0880	0.0920	0.0934	0.0942	0.0947	0.0950	0.0954	0.0956	0.0959	0.0960	0.0962	0.0963	t
0.15	s	0.0833	0.0872	0.0904	0.0915	0.0922	0.0925	0.0928	0.0930	0.0932	0.0934	0.0934	0.0936	0.0937	t

TABLE 19.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.

TOTAL LOAD 1, UNIFORMLY DISTRIBUTED.

CASE I(b).—UNSYMMETRICAL, SHARPLY CURVED HAUNCH.

$b = \frac{I}{I'}$

Values of a :	Haunch at left support.	VALUES OF b :												Haunch at right support.	
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
1.00	s	0.0833	0.0864	0.0906	0.0929	0.0941	0.0952	0.0958	0.0966	0.0974	0.0976	0.0981	0.0985	0.0991	t
	t	0.0833	0.0809	0.0783	0.0772	0.0767	0.0764	0.0762	0.0759	0.0757	0.0756	0.0755	0.0753	0.0752	s
0.50	s	0.0833	0.0885	0.0939	0.0963	0.0976	0.0984	0.0988	0.0995	0.1000	0.1002	0.1006	0.1008	0.1012	t
	t	0.0833	0.0832	0.0831	0.0830	0.0830	0.0830	0.0830	0.0830	0.0830	0.0830	0.0830	0.0830	0.0830	s
0.40	s	0.0833	0.0887	0.0940	0.0962	0.0973	0.0979	0.0985	0.0990	0.0993	0.0997	0.1000	0.1003	0.1006	t
	t	0.0833	0.0836	0.0836	0.0837	0.0837	0.0837	0.0837	0.0837	0.0838	0.0838	0.0838	0.0838	0.0838	s
0.35	s	0.0833	0.0887	0.0938	0.0958	0.0969	0.0975	0.0979	0.0984	0.0988	0.0991	0.0994	0.0996	0.0999	t
	t	0.0833	0.0836	0.0837	0.0838	0.0838	0.0838	0.0839	0.0839	0.0839	0.0839	0.0839	0.0839	0.0839	s
0.30	s	0.0833	0.0885	0.0934	0.0952	0.0962	0.0968	0.0971	0.0975	0.0980	0.0981	0.0984	0.0986	0.0987	t
	t	0.0833	0.0836	0.0838	0.0838	0.0839	0.0839	0.0840	0.0840	0.0840	0.0840	0.0840	0.0840	0.0841	s
0.25	s	0.0833	0.0882	0.0926	0.0941	0.0951	0.0956	0.0960	0.0963	0.0966	0.0969	0.0970	0.0971	0.0973	t
	t	0.0833	0.0836	0.0838	0.0839	0.0839	0.0839	0.0839	0.0840	0.0840	0.0840	0.0840	0.0840	0.0840	s
0.20	s	0.0833	0.0877	0.0915	0.0929	0.0936	0.0940	0.0943	0.0946	0.0949	0.0950	0.0953	0.0954	0.0956	t
	t	0.0833	0.0836	0.0837	0.0838	0.0838	0.0838	0.0839	0.0839	0.0839	0.0839	0.0839	0.0839	0.0838	s

TABLE 20.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.

TOTAL LOAD 1, UNIFORMLY DISTRIBUTED.

CASE II(a).—SYMMETRICAL, STRAIGHT HAUNCHES.

$b = \frac{I}{I'}$

Values of a :		VALUES OF b :													
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
0.50	s	0.0833	0.0885	0.0951	0.0986	0.1008	0.1025	0.1039	0.1055	0.1074	0.1085	0.1098	0.1115	0.1136	t
0.40	s	0.0833	0.0894	0.0967	0.1006	0.1031	0.1049	0.1063	0.1078	0.1099	0.1110	0.1123	0.1136	0.1154	t
0.35	s	0.0833	0.0894	0.0968	0.1005	0.1030	0.1048	0.1060	0.1075	0.1093	0.1102	0.1114	0.1126	0.1142	t
0.30	s	0.0833	0.0893	0.0930	0.0999	0.1020	0.1037	0.1048	0.1061	0.1077	0.1086	0.1096	0.1108	0.1120	t
0.25	s	0.0833	0.0890	0.0954	0.0986	0.1006	0.1018	0.1030	0.1040	0.1054	0.1062	0.1070	0.1080	0.1091	t
0.20	s	0.0833	0.0884	0.0940	0.0967	0.0984	0.0995	0.1003	0.1013	0.1024	0.1030	0.1037	0.1044	0.1054	t
0.15	s	0.0833	0.0876	0.0921	0.0942	0.0955	0.0964	0.0970	0.0978	0.0986	0.0991	0.0996	0.1002	0.1010	t

TABLE 21.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
TOTAL LOAD 1, UNIFORMLY DISTRIBUTED.
CASE II(b).—UNSYMMETRICAL, STRAIGHT HAUNCH.

$$b = \frac{I}{I'}$$

Values of a :	Haunch at left support.	VALUES OF b :													Haunch at right support.
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
1.00	s	0.0833	0.0838	0.0884	0.0899	0.0905	0.0908	0.0910	0.0910	0.0909	0.0905	0.0900	0.0894	0.0885	t
	t	0.0833	0.0805	0.0755	0.0722	0.0701	0.0683	0.0666	0.0645	0.0622	0.0607	0.0588	0.0563	0.0527	s
0.50	s	0.0833	0.0885	0.0947	0.0976	0.0993	0.1006	0.1014	0.1025	0.1035	0.1040	0.1047	0.1051	0.1058	t
	t	0.0833	0.0831	0.0826	0.0822	0.0819	0.0817	0.0815	0.0812	0.0810	0.0808	0.0805	0.0803	0.0799	s
0.40	s	0.0833	0.0889	0.0935	0.0985	0.1006	0.1018	0.1028	0.1038	0.1051	0.1057	0.1065	0.1073	0.1082	t
	t	0.0833	0.0835	0.0835	0.0834	0.0834	0.0833	0.0833	0.0832	0.0832	0.0831	0.0830	0.0830	0.0828	s
0.35	s	0.0833	0.0890	0.0954	0.0985	0.1004	0.1016	0.1028	0.1038	0.1052	0.1057	0.1065	0.1073	0.1082	t
	t	0.0833	0.0835	0.0837	0.0838	0.0838	0.0838	0.0838	0.0838	0.0838	0.0838	0.0838	0.0837	0.0836	s
0.30	s	0.0833	0.0889	0.0951	0.0981	0.1000	0.1012	0.1021	0.1032	0.1044	0.1048	0.1057	0.1065	0.1075	t
	t	0.0833	0.0836	0.0839	0.0840	0.0841	0.0841	0.0841	0.0841	0.0841	0.0842	0.0842	0.0842	0.0842	s
0.25	s	0.0833	0.0886	0.0944	0.0972	0.0988	0.0999	0.1009	0.1018	0.1028	0.1034	0.1042	0.1049	0.1057	t
	t	0.0833	0.0836	0.0839	0.0840	0.0841	0.0842	0.0842	0.0842	0.0842	0.0843	0.0843	0.0843	0.0844	s
0.20	s	0.0833	0.0881	0.0933	0.0956	0.0972	0.0981	0.0988	0.0998	0.1007	0.1012	0.1017	0.1024	0.1031	t
	t	0.0833	0.0836	0.0838	0.0839	0.0840	0.0841	0.0841	0.0841	0.0841	0.0841	0.0842	0.0842	0.0843	s

TABLE 22.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
TOTAL LOAD 1, UNIFORMLY DISTRIBUTED.
CASE III(a).—SYMMETRICAL, PARABOLIC HAUNCHES.

$$b = \frac{I}{I'}$$

Values of a :		VALUES OF b :													
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	0.02	
0.50	s	0.0833	0.0888	0.0954	0.0988	0.1012	0.1028	0.1040	0.1055	0.1073	0.1084	0.1096	0.1109	0.1126	t
0.40	s	0.0833	0.0887	0.0951	0.0983	0.1005	0.1021	0.1031	0.1044	0.1062	0.1070	0.1081	0.1095	0.1110	t
0.35	s	0.0833	0.0885	0.0946	0.0976	0.0997	0.1010	0.1021	0.1034	0.1048	0.1056	0.1066	0.1078	0.1094	t
0.30	s	0.0833	0.0882	0.0938	0.0965	0.0983	0.0996	0.1006	0.1017	0.1031	0.1038	0.1047	0.1058	0.1071	t
0.25	s	0.0833	0.0878	0.0928	0.0952	0.0968	0.0979	0.0988	0.0996	0.1008	0.1015	0.1023	0.1032	0.1043	t
0.20	s	0.0833	0.0871	0.0914	0.0935	0.0948	0.0957	0.0964	0.0972	0.0982	0.0987	0.0994	0.1001	0.1011	t
0.15	s	0.0833	0.0864	0.0898	0.0914	0.0923	0.0932	0.0938	0.0944	0.0951	0.0955	0.0961	0.0966	0.0974	t

TABLE 23.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
TOTAL LOAD 1, UNIFORMLY DISTRIBUTED.
CASE III(b).—UNSYMMETRICAL, PARABOLIC HAUNCH.

$$b = \frac{I}{I'}$$

Values of a :	Haunch at left support.	VALUES OF b :												Haunch at right support.	
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03		0.02
1.00	s	0.0833	0.0878	0.0916	0.0935	0.0945	0.0952	0.0956	0.0961	0.0962	0.0962	0.0962	0.0961	0.0955	t
	t	0.0833	0.0813	0.0792	0.0773	0.0761	0.0750	0.0741	0.0729	0.0715	0.0704	0.0692	0.0676	0.0653	s
0.50	s	0.0833	0.0866	0.0946	0.0974	0.0992	0.1005	0.1014	0.1024	0.1036	0.1049	0.1051	0.1060	0.1068	t
	t	0.0833	0.0835	0.0835	0.0834	0.0834	0.0833	0.0832	0.0831	0.0831	0.0830	0.0829	0.0828	0.0827	s
0.40	s	0.0833	0.0884	0.0941	0.0969	0.0987	0.1000	0.1008	0.1019	0.1032	0.1038	0.1046	0.1055	0.1066	t
	t	0.0833	0.0836	0.0837	0.0838	0.0839	0.0839	0.0839	0.0839	0.0839	0.0839	0.0839	0.0839	0.0838	s
0.35	s	0.0833	0.0882	0.0937	0.0964	0.0980	0.0992	0.1000	0.1011	0.1023	0.1030	0.1037	0.1046	0.1056	t
	t	0.0833	0.0836	0.0838	0.0839	0.0840	0.0840	0.0840	0.0840	0.0841	0.0841	0.0841	0.0842	0.0842	s
0.30	s	0.0833	0.0879	0.0930	0.0955	0.0970	0.0982	0.0990	0.0999	0.1011	0.1018	0.1024	0.1032	0.1042	t
	t	0.0833	0.0836	0.0838	0.0839	0.0840	0.0840	0.0840	0.0840	0.0841	0.0841	0.0842	0.0842	0.0842	s
0.25	s	0.0833	0.0875	0.0922	0.0944	0.0957	0.0967	0.0975	0.0984	0.0993	0.0999	0.1006	0.1014	0.1023	t
	t	0.0833	0.0835	0.0837	0.0838	0.0839	0.0840	0.0840	0.0841	0.0841	0.0841	0.0842	0.0842	0.0842	s
0.20	s	0.0833	0.0869	0.0910	0.0930	0.0940	0.0949	0.0955	0.0963	0.0973	0.0977	0.0982	0.0990	0.0998	t
	t	0.0833	0.0835	0.0837	0.0837	0.0838	0.0839	0.0839	0.0839	0.0839	0.0840	0.0840	0.0841	0.0841	s

TABLE 24.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
TOTAL TRIANGULAR LOAD 1. ALL CASES.

SYMMETRICAL BEAM. $b = \frac{I}{I'}$.

Values of a :	Load maximum at left support.	UNIFORM MOMENT OF INERTIA.	CASE I (a).— SYMMETRICAL, SHARPLY CURVED HAUNCHES.					CASE II (a).— SYMMETRICAL, STRAIGHT HAUNCHES.				CASE III (a).— SYMMETRICAL, PARABOLIC HAUNCHES.				Load maximum at right support.
			Values of b :					Values of b :				Values of b :				
			1.00	0.20	0.10	0.05	0.03	0.20	0.10	0.05	0.03	0.20	0.10	0.05	0.03	
0.50	s		0.0889	0.1019	0.1047	0.1064	0.1072	0.1028	0.1075	0.1114	0.1141	0.1033	0.1079	0.1118	0.1139	t
	t		0.0778	0.0926	0.0959	0.0978	0.0987	0.0943	0.1003	0.1054	0.1099	0.0944	0.1000	0.1049	0.1078	s
0.40	s		0.0889	0.1022	0.1047	0.1062	0.1068	0.1047	0.1097	0.1138	0.1160	0.1030	0.1074	0.1107	0.1129	t
	t		0.0778	0.0928	0.0957	0.0973	0.0980	0.0964	0.1028	0.1082	0.1112	0.0937	0.0989	0.1032	0.1060	s
0.35	s		0.0889	0.1018	0.1042	0.1055	0.1061	0.1047	0.1096	0.1138	0.1153	0.1024	0.1065	0.1097	0.1117	t
	t		0.0778	0.0922	0.0949	0.0963	0.0969	0.0962	0.1022	0.1069	0.1097	0.0927	0.0975	0.1016	0.1040	s
0.30	s		0.0889	0.1013	0.1034	0.1045	0.1050	0.1043	0.1088	0.1122	0.1141	0.1015	0.1053	0.1088	0.1102	t
	t		0.0778	0.0913	0.0936	0.0948	0.0953	0.0954	0.1008	0.1049	0.1075	0.0915	0.0959	0.0993	0.1015	s
0.25	s		0.0889	0.1003	0.1021	0.1030	0.1034	0.1034	0.1075	0.1104	0.1120	0.1004	0.1038	0.1064	0.1079	t
	t		0.0778	0.0909	0.0919	0.0929	0.0933	0.0938	0.0985	0.1021	0.1040	0.0900	0.0937	0.0967	0.0985	s

TABLE 25.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.

TOTAL TRIANGULAR LOAD 1.

CASE I(b).—UNSYMMETRICAL, SHARPLY CURVED HAUNCH.

Diagram of a beam of length l with a triangular load increasing from left to right. The load at the right end is b . The load at the left end is 0. The load at a distance al from the left end is $a \cdot b$. The load at a distance $l-al$ from the right end is $(1-a) \cdot b$.

Diagram of a beam of length l with a triangular load decreasing from left to right. The load at the left end is b' . The load at the right end is 0. The load at a distance $l-al$ from the left end is $(1-a) \cdot b'$. The load at a distance al from the right end is $a \cdot b'$.

LOAD MAXIMUM AT HAUNCH.

LOAD ZERO AT HAUNCH.

Values of a :		LOAD MAXIMUM AT HAUNCH.						LOAD ZERO AT HAUNCH.							
		Values of b :					Values of b :								
		Haunch at left support.	1.00	0.20	0.10	0.05	0.03	Haunch at left support.	1.00	0.20	0.10	0.05	0.03	Haunch at right support.	
1.00	s		0.0889	0.0953	0.0973	0.0984	0.0989	t	s	0.0778	0.0906	0.0945	0.0970	0.0980	t
	t		0.0778	0.0697	0.0682	0.0675	0.0673	s	t	0.0889	0.0848	0.0841	0.0838	0.0836	s
0.50	s		0.0889	0.0992	0.1012	0.1025	0.1029	t	s	0.0778	0.0934	0.0965	0.0982	0.0989	t
	t		0.0778	0.0762	0.0760	0.0759	0.0758	s	t	0.0889	0.0899	0.0901	0.0901	0.0901	s
0.40	s		0.0889	0.0996	0.1018	0.1028	0.1033	t	s	0.0778	0.0927	0.0954	0.0967	0.0973	t
	t		0.0778	0.0773	0.0771	0.0771	0.0771	s	t	0.0889	0.0901	0.0902	0.0903	0.0904	s
0.35	s		0.0889	0.0998	0.1017	0.1028	0.1031	t	s	0.0778	0.0919	0.0944	0.0956	0.0962	t
	t		0.0778	0.0776	0.0776	0.0775	0.0775	s	t	0.0889	0.0901	0.0902	0.0902	0.0903	s
0.30	s		0.0889	0.0996	0.1014	0.1023	0.1027	t	s	0.0778	0.0908	0.0930	0.0940	0.0946	t
	t		0.0778	0.0778	0.0778	0.0778	0.0778	s	t	0.0889	0.0899	0.0899	0.0900	0.0902	s
0.25	s		0.0889	0.0990	0.1006	0.1015	0.1018	t	s	0.0778	0.0894	0.0913	0.0922	0.0926	t
	t		0.0778	0.0780	0.0780	0.0780	0.0781	s	t	0.0889	0.0897	0.0898	0.0898	0.0899	s

TABLE 26.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.

TOTAL TRIANGULAR LOAD 1.

CASE II(b).—UNSYMMETRICAL, STRAIGHT HAUNCH.

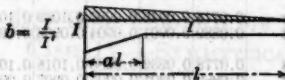

															
		LOAD MAXIMUM AT HAUNCH.								LOAD ZERO AT HAUNCH.					
Values of a:		Values of b:					Haunch at right support.			Values of b:					Haunch at right support.
Haunch at left support.		1.00	0.20	0.10	0.05	0.03		Haunch at left support.		1.00	0.20	0.10	0.05	0.03	
1.00	s	0.0889	0.0919	0.0910	0.0892	0.0871	t	s	0.0778	0.0877	0.0901	0.0914	0.0917	t	
	t	0.0778	0.0645	0.0588	0.0520	0.0476	s	t	0.0889	0.0803	0.0751	0.0694	0.0650	s	
0.50	s	0.0889	0.0988	0.1007	0.1014	0.1015	t	s	0.0778	0.0934	0.1023	0.1067	0.1090	t	
	t	0.0778	0.0746	0.0738	0.0720	0.0712	s	t	0.0889	0.0898	0.0897	0.0896	0.0894	s	
0.40	s	0.0889	0.1007	0.1037	0.1054	0.1062	t	s	0.0778	0.0933	0.1021	0.1064	0.1088	t	
	t	0.0778	0.0765	0.0759	0.0753	0.0750	s	t	0.0889	0.0903	0.0907	0.0907	0.0908	s	
0.35	s	0.0889	0.1014	0.1046	0.1065	0.1074	t	s	0.0778	0.0937	0.1011	0.1051	0.1073	t	
	t	0.0778	0.0772	0.0768	0.0766	0.0763	s	t	0.0889	0.0904	0.0908	0.0910	0.0911	s	
0.30	s	0.0889	0.1016	0.1049	0.1068	0.1079	t	s	0.0778	0.0946	0.0996	0.1030	0.1051	t	
	t	0.0778	0.0777	0.0775	0.0774	0.0773	s	t	0.0889	0.0903	0.0906	0.0909	0.0911	s	
0.25	s	0.0889	0.1014	0.1046	0.1066	0.1078	t	s	0.0778	0.0931	0.0974	0.1004	0.1023	t	
	t	0.0778	0.0780	0.0780	0.0780	0.0780	s	t	0.0889	0.0901	0.0904	0.0906	0.0908	s	

TABLE 27.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA.
TOTAL TRIANGULAR LOAD 1.
CASE III(b).—UNSYMMETRICAL, PARABOLIC HAUNCH.

LOAD ZERO AT HAUNCH.

LOAD ZERO AT HAUNCH.

Values of b :

Values of b :

Values of b :

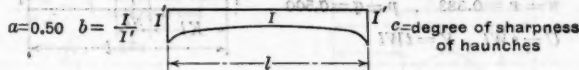
Values of b :

TABLE 27.—LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA
TOTAL TRIANGULAR LOAD 1.
CASE III(b).—UNSYMMETRICAL, PARABOLIC HAUNCH.

Values of a :		LOAD MAXIMUM AT HAUNCH.						LOAD ZERO AT HAUNCH.						
		Haunch at left support.	Values of b :					Haunch at left support.	Values of b :					
			1.00	0.20	0.10	0.05	0.03		1.00	0.20	0.10	0.05	0.03	
1.00	s	s	0.0889	0.0652	0.0652	0.0689	0.0922	t	s	0.0778	0.0920	0.0961	0.0989	0.1000
	t	t	0.0778	0.0693	0.0653	0.0610	0.0579	s	t	0.0889	0.0852	0.0828	0.0797	0.0772
0.50	s	s	0.0889	0.1005	0.1031	0.1048	0.1055	t	s	0.0778	0.0946	0.0997	0.1039	0.1064
	t	t	0.0778	0.0767	0.0761	0.0755	0.0751	s	t	0.0889	0.0901	0.0904	0.0906	0.0907
0.40	s	s	0.0889	0.1007	0.1037	0.1058	0.1069	t	s	0.0778	0.0932	0.0980	0.1018	0.1042
	t	t	0.0778	0.0775	0.0774	0.0770	0.0769	s	t	0.0889	0.0901	0.0905	0.0907	0.0909
0.35	s	s	0.0889	0.1006	0.1036	0.1057	0.1069	t	s	0.0778	0.0923	0.0967	0.1002	0.1023
	t	t	0.0778	0.0779	0.0777	0.0775	0.0775	s	t	0.0889	0.0901	0.0903	0.0906	0.0908
0.30	s	s	0.0889	0.1001	0.1030	0.1053	0.1063	t	s	0.0778	0.0910	0.0949	0.0982	0.1000
	t	t	0.0778	0.0780	0.0780	0.0779	0.0779	s	t	0.0889	0.0899	0.0902	0.0904	0.0905
0.25	s	s	0.0889	0.0994	0.1021	0.1043	0.1054	t	s	0.0778	0.0895	0.0929	0.0957	0.0974
	t	t	0.0778	0.0780	0.0781	0.0781	0.0781	s	t	0.0889	0.0897	0.0899	0.0901	0.0903

TABLE 28.—BEAM AND LOAD COEFFICIENTS FOR BEAMS OF VARYING MOMENT OF INERTIA. BEAM COEFFICIENTS.

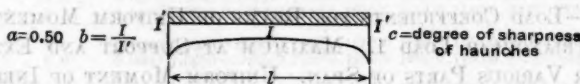
CASE IV.—SYMMETRICAL, SHARPLY CURVED HAUNCHES, OF VARYING DEGREES OF SHARPNESS.



Values of c:		VALUES OF b:												
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	
0.5	u	0.333	0.351	0.375	0.388	0.396	0.401	0.405	0.409	0.413	0.416	0.418	0.420	v
	p	0.500	0.367	0.267	0.233	0.217	0.207	0.200	0.193	0.187	0.183	0.180	0.177	q
1.0	u	0.333	0.354	0.378	0.389	0.396	0.399	0.402	0.404	0.407	0.409	0.410	0.412	v
	p	0.500	0.400	0.325	0.300	0.288	0.280	0.275	0.270	0.265	0.263	0.260	0.257	q
2.0	u	0.333	0.354	0.374	0.382	0.386	0.389	0.390	0.392	0.394	0.395	0.396	0.397	v
	p	0.500	0.433	0.383	0.367	0.358	0.353	0.350	0.347	0.343	0.342	0.340	0.338	q
3.0	u	0.333	0.352	0.369	0.375	0.378	0.380	0.382	0.383	0.385	0.385	0.386	0.387	v
	p	0.500	0.450	0.412	0.400	0.394	0.390	0.388	0.385	0.382	0.381	0.380	0.379	q
5.0	u	0.333	0.348	0.361	0.365	0.368	0.369	0.370	0.371	0.372	0.373	0.373	0.374	v
	p	0.500	0.467	0.442	0.433	0.429	0.427	0.425	0.423	0.422	0.421	0.420	0.419	q
10.0	u	0.333	0.343	0.351	0.353	0.355	0.356	0.356	0.357	0.357	0.358	0.358	0.358	v
	p	0.500	0.482	0.468	0.464	0.461	0.460	0.459	0.458	0.457	0.457	0.456	0.456	q

TABLE 29.—LOAD COEFFICIENTS. TOTAL LOAD 1, UNIFORMLY DISTRIBUTED.*

CASE IV.—SYMMETRICAL, SHARPLY CURVED HAUNCHES, OF VARYING DEGREE OF SHARPNESS.



Values of c:		VALUES OF b:												
		1.00	0.60	0.30	0.20	0.15	0.12	0.10	0.08	0.06	0.05	0.04	0.03	
0.5	s	0.0833	0.0877	0.0938	0.0969	0.0989	0.1002	0.1012	0.1022	0.1033	0.1039	0.1045	0.1051	t
1.0	s	0.0833	0.0885	0.0946	0.0972	0.0987	0.0997	0.1004	0.1011	0.1018	0.1022	0.1026	0.1030	t
2.0	s	0.0833	0.0885	0.0935	0.0955	0.0965	0.0972	0.0976	0.0981	0.0985	0.0988	0.0990	0.0993	t
3.0	s	0.0833	0.0880	0.0922	0.0938	0.0946	0.0951	0.0954	0.0958	0.0961	0.0963	0.0965	0.0967	t
5.0	s	0.0833	0.0871	0.0902	0.0913	0.0919	0.0923	0.0925	0.0928	0.0930	0.0931	0.0933	0.0934	t
10.0	s	0.0833	0.0858	0.0877	0.0884	0.0887	0.0889	0.0890	0.0892	0.0893	0.0894	0.0895	0.0895	t

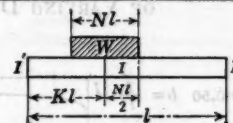
* To obtain load coefficients for other loadings, see p. 689.

TABLE 30.—LOAD COEFFICIENTS FOR BEAMS OF UNIFORM MOMENT OF INERTIA.
TOTAL LOAD 1. UNIFORMLY DISTRIBUTED OVER VARIOUS PARTS
OF THE SPAN. UNIFORM MOMENT OF INERTIA.

$b = \frac{I}{I'}, = 1.000$

$u = v = 0.333 \quad , \quad p = q = 0.500$

$U = sWl \quad , \quad V = iWl$



s

Values of K:

VALUES OF N:

Values of K:

t

	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	
0.05	0.0801	0.95
0.10	0.0562	0.0540	0.90
0.15	0.0779	0.0758	0.0722	0.85
0.20	0.0953	0.0933	0.0900	0.0853	0.80
0.25	0.1088	0.1069	0.1037	0.0994	0.0938	0.75
0.30	0.1184	0.1167	0.1138	0.1097	0.1044	0.0980	0.70
0.35	0.1246	0.1230	0.1202	0.1165	0.1116	0.1056	0.0986	0.65
0.40	0.1275	0.1260	0.1235	0.1200	0.1155	0.1100	0.1035	0.0960	0.60
0.45	0.1274	0.1260	0.1238	0.1205	0.1164	0.1114	0.1054	0.0985	0.0900	0.55
0.50	0.1246	0.1233	0.1212	0.1183	0.1146	0.1100	0.1046	0.0983	0.0912	0.0833	0.50
0.55	0.1192	0.1181	0.1162	0.1136	0.1102	0.1061	0.1012	0.0956	0.0892	0.45
0.60	0.1117	0.1107	0.1090	0.1067	0.1037	0.1000	0.0957	0.0907	0.40
0.65	0.1021	0.1012	0.0998	0.0977	0.0951	0.0919	0.0881	0.35
0.70	0.0908	0.0900	0.0887	0.0870	0.0848	0.0820	0.30
0.75	0.0779	0.0773	0.0762	0.0748	0.0729	0.25
0.80	0.0638	0.0633	0.0625	0.0613	0.20
0.85	0.0488	0.0484	0.0477	0.15
0.90	0.0329	0.0327	0.10
0.95	0.0166	0.05

s

Values of K:

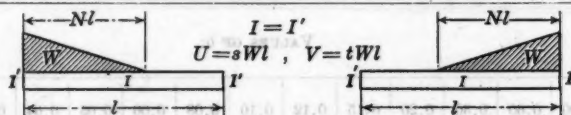
VALUES OF N:

Values of K:

t

TABLE 31.—LOAD COEFFICIENTS FOR BEAMS OF UNIFORM MOMENT OF INERTIA.
TOTAL TRIANGULAR LOAD 1. MAXIMUM AT SUPPORT AND EXTENDING
OVER VARIOUS PARTS OF SPAN. UNIFORM MOMENT OF INERTIA.

$I = I'$
 $U = sWl, \quad V = tWl$



Load maxi- mum at left support.	VALUES OF N:										Load maxi- mum at right support.
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	
s	0.0206	0.0381	0.0526	0.0643	0.0736	0.0805	0.0853	0.0881	0.0893	0.0889	t
t	0.0111	0.0220	0.0324	0.0423	0.0514	0.0595	0.0663	0.0718	0.0757	0.0778	s

TABLE 32.—MOMENT COEFFICIENTS FOR SIMPLY SUPPORTED BEAM AND LOAD
COEFFICIENTS FOR BEAMS OF UNIFORM MOMENT OF INERTIA. TOTAL
LOAD 1. UNIFORM, TRAPEZOIDAL, AND TRIANGULAR
DISTRIBUTION. UNIFORM MOMENT OF INERTIA.

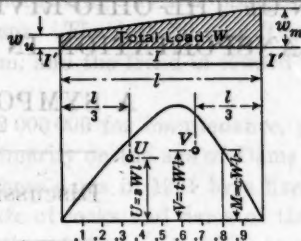
$$\frac{w_m}{w_u} = \begin{cases} \text{ratio of max. to min.} \\ \text{intensity of load} \end{cases}$$

$$b = \frac{l}{l'}, = \text{one}$$

$$u = v = 0.333$$

$$p = q = 0.500$$

$$U = sWl, V = tWl$$



Type of loading :	Values of $\frac{W_m}{W_u}$:	LOAD MAXIMUM AT RIGHT SUPPORT, READ DOWN.										Load coefficient.	
		Coefficient for simple beam moment, at Point.									s. t.		
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9			
Uniform.	1.0	0.0450	0.0800	0.1050	0.1200	0.1250	0.1200	0.1050	0.0800	0.0450	0.0833	0.0833	
Trapezoidal.	1.1	0.0444	0.0792	0.1044	0.1196	0.1250	0.1204	0.1056	0.0808	0.0456	0.0830	0.0836	
	1.2	0.0439	0.0788	0.1038	0.1192	0.1250	0.1208	0.1062	0.0815	0.0461	0.0828	0.0839	
	1.3	0.0434	0.0779	0.1031	0.1190	0.1250	0.1210	0.1064	0.0821	0.0465	0.0826	0.0840	
	1.4	0.0430	0.0772	0.1026	0.1186	0.1250	0.1214	0.1072	0.0826	0.0470	0.0824	0.0842	
	1.5	0.0425	0.0768	0.1021	0.1184	0.1250	0.1216	0.1073	0.0832	0.0474	0.0821	0.0845	
	1.6	0.0421	0.0762	0.1018	0.1181	0.1250	0.1219	0.1082	0.0838	0.0478	0.0820	0.0846	
	1.8	0.0415	0.0754	0.1010	0.1178	0.1250	0.1222	0.1090	0.0846	0.0484	0.0818	0.0849	
	2.0	0.0410	0.0746	0.1002	0.1174	0.1250	0.1226	0.1096	0.0854	0.0490	0.0815	0.0851	
	2.25	0.0404	0.0739	0.0996	0.1169	0.1250	0.1230	0.1104	0.0861	0.0496	0.0811	0.0855	
	2.50	0.0399	0.0731	0.0990	0.1165	0.1250	0.1234	0.1110	0.0869	0.0501	0.0809	0.0858	
	2.75	0.0394	0.0725	0.0985	0.1162	0.1250	0.1238	0.1115	0.0875	0.0506	0.0806	0.0860	
	3.0	0.0390	0.0720	0.0980	0.1160	0.1250	0.1240	0.1120	0.0880	0.0510	0.0805	0.0861	
	3.5	0.0384	0.0711	0.0972	0.1155	0.1250	0.1244	0.1128	0.0889	0.0516	0.0802	0.0864	
	4.0	0.0378	0.0704	0.0966	0.1152	0.1250	0.1248	0.1134	0.0896	0.0521	0.0800	0.0866	
	4.5	0.0374	0.0698	0.0961	0.1149	0.1250	0.1251	0.1139	0.0901	0.0526	0.0798	0.0869	
	5.0	0.0370	0.0694	0.0956	0.1146	0.1250	0.1254	0.1144	0.0906	0.0530	0.0796	0.0870	
	6.0	0.0364	0.0685	0.0950	0.1142	0.1250	0.1258	0.1150	0.0915	0.0535	0.0794	0.0874	
7.0	0.0360	0.0680	0.0945	0.1140	0.1250	0.1260	0.1155	0.0920	0.0540	0.0791	0.0875		
8.0	0.0356	0.0675	0.0941	0.1138	0.1250	0.1262	0.1159	0.0925	0.0542	0.0790	0.0876		
9.0	0.0354	0.0671	0.0938	0.1136	0.1250	0.1264	0.1161	0.0928	0.0546	0.0789	0.0878		
10.0	0.0351	0.0669	0.0935	0.1135	0.1250	0.1265	0.1165	0.0931	0.0548	0.0788	0.0879		
15.0	0.0345	0.0660	0.0928	0.1130	0.1250	0.1270	0.1172	0.0940	0.0555	0.0785	0.0881		
20.0	0.0341	0.0655	0.0924	0.1128	0.1250	0.1272	0.1176	0.0945	0.0559	0.0784	0.0884		
40.0	0.0335	0.0648	0.0916	0.1124	0.1250	0.1276	0.1184	0.0952	0.0564	0.0780	0.0886		
Triangle.	Infinity.	0.0330	0.0640	0.0910	0.1120	0.1250	0.1280	0.1190	0.0960	0.0570	0.0778	0.0889	
Type of loading :	Values of $\frac{W_m}{W_u}$:	Coefficient for simple beam moment at Point.									Load coefficient.		
		0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	t.	s.	
LOAD MAXIMUM AT LEFT SUPPORT, READ UP.													

RELATION OF THE OHIO RIVER AND ITS TRIBUTARIES TO TRANSPORTATION IN THE UNITED STATES

A SYMPOSIUM

Discussion*

By C. W. KUTZ, M. Am. Soc. C. E.†

C. W. KUTZ,‡ M. Am. Soc. C. E. (by letter).§—A review of the papers presented in this Symposium and the discussion thereon shows widely varying conclusions as to the value of the Ohio River as a transportation agency notwithstanding the fact that nearly all the economic comparisons are based on a consideration of the same basic data.

The principal points of disagreement may be grouped as follows:

- 1.—The annual charges on the waterway when the present project is completed;
- 2.—The benefits as measured by the difference between rail rates and water haulage costs when applied to existing tonnage; and
- 3.—Prospective tonnage.

Annual Charges.—The annual charges depend on first cost, cost of maintenance and operation, and depreciation. For purposes of comparison with a privately owned transportation agency, an item for lost taxes should also receive consideration. In the writer's paper|| the cost of this project was given as \$103 000 000; Mr. Alfred¶ used \$110 000 000. The present project was adopted in 1910. There was expended under previous projects, including the amount for the purchase of the Louisville and Portland Canal, approximately \$17 500 000. This amount also includes the cost of Dams Nos. 1 and 2—since replaced. If all this amount, as well as the cost of Emsworth Dam which replaced Dams Nos. 1 and 2, is considered as part of the first cost, the total will aggregate approximately \$110 000 000. This total also includes the entire cost of the proposed power navigation dam at Louisville, Ky., but, as the Power Company will pay annually interest and depreciation on the difference between the cost of the navigation dam and the cost of the power-navigation dam, this difference, amounting to approximately \$1 500 000, should be deducted from the total, leaving the net cost of the Ohio River Canalization Project as

* Discussion continued from February, 1926, *Proceedings*.

† Author's closure.

‡ Col., Corps of Engrs., U. S. A., Cincinnati, Ohio.

§ Received by the Secretary, February 23, 1926.

|| *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1645.

¶ *Loc. cit.*, p. 1664.

\$108 500 000. If the cost is limited to that of work now used and useful in the interest of navigation, the total of \$103 000 000 would be more nearly correct.

The writer used a 4% rate in determining interest charges, as did Mr. Alfred. Mr. Begien* used 4½ per cent. The last 30-year Treasury bonds, bearing 4% interest, sold at a premium, and the trend is toward lower, rather than higher, rates.

Mr. Alfred† assumes an item of \$2 000 000 for maintenance, plus 1½% for depreciation, the latter being based primarily on the age of Dams Nos. 1 and 2 when replaced. The replacement of these dams in 1921 by a fixed dam is not a fair index of the length of useful life of locks and dams of the Ohio River type, as their replacement was primarily a betterment, for the purpose of improving conditions in Pittsburgh Harbor. The replacement of Dam No. 3 and Dam No. 4 by a single fixed dam is also under consideration and for a similar reason.

The movable parts of these structures such as wickets, gates, valves, etc., have only a limited life and require replacement from time to time, but this is not equally true of the concrete walls and foundations. Based on experience, the cost of maintaining and operating Ohio River locks and dams, including the average annual cost of replacing depreciable parts, was estimated at \$40 000 per lock and dam, or \$2 000 000 for the entire river. A reconsideration of this item leads to the belief that it is too small and that the average annual cost during a long period will be approximately \$2 250 000.

If this type of waterway is to be superseded eventually by some superior form of transportation agency, as were the old canals of 100 years ago, an economic comparison should properly include in the annual charges an item whereby the entire first cost will be amortized during the useful life of the project. Although this supposition might be applied with almost equal force to the privately owned railroads, to provide against such a contingency for the waterway only it is proposed to assume a life of 75 years and set up as part of the annual charges an item for depreciation of 0.22% of the first cost, an amount which is sufficient when compounded at 4% to amortize the entire first cost during the assumed life.

Mr. Alfred‡ includes in his estimate of annual charges an item of taxes of 1.46%, the average paid by the railroads of the United States in 1923. As to the propriety of including such an item, the following is quoted from a report made by J. M. Clark, Associate Professor of Economics, Chicago University, a number of years ago, in connection with preliminary investigation of canal routes connecting Lake Erie with the Ohio River:

"Another question is whether taxes should be charged on the public investment, on the ground that if that capital had been left in the market to be used in private investments, the Government would have received taxes on it. The principle is sound: a public investment which yields the Government only what it has to pay out again on its bonds, leaves the Government poorer than if the same funds had gone into a private investment on which the Gov-

* *Proceedings, Am. Soc. C. E.*, November, 1925, *Papers and Discussions*, p. 1885.

† *Loc. cit.*, October, 1925, *Papers and Discussions*, p. 1667.

ernment receives taxes. The fact that the bulk of the taxes would not go to the Federal Government does not affect this principle; it merely makes the loss fall on the States.

"There is, however, one real question. Does an investment of this sort always result in reducing private investments by just the same amount? 'Orthodox' economics would say yes: but, in cases where the financing can be done partly through credit expansion, it seems probable that the correct answer is no; \$100 000 000 of Government funds invested in this way would displace some private investments, but less than \$100 000 000. It would depend on whether the credit mechanism had some reserve lending power at the time the loans were floated, and also on the existence, at the time, of a certain reserve of labor and plant capacity which could be drawn upon without diverting labor from other things. These conditions might or might not prevail. * * * If taxes are counted at the usual rate on one-half of the investment rather than on the whole, all reasonable allowance will have been made for the elasticity of the industrial system."

Although there is room for a difference of opinion as to the inclusion of such an item, the writer believes that full justice will be done the railroads by the adoption of Professor Clark's views. Accordingly, in the revised summary of annual charges, taxes will be included at 0.73%—one-half the average rate paid by the railroads in 1923, which is equivalent to the full rate on one-half the investment. The total annual charges on completion of the project will then become:

Interest at 4% on \$108 500 000.....	\$4 340 000
Taxes at 0.73%.....	792 050
Depreciation (obsolescence) at 0.22%.....	233 400
Maintenance and operation (including replacement of depreciable parts)	2 250 000
Total	\$7 615 450

This revised total is \$1 500 000 greater than the annual charges given by the writer in Table 3;* \$600 000 greater than the estimate of Mr. Begien,† but still \$2 040 000 less than that of Mr. Alfred.‡ Measured by the expenditures to date the project is 80% completed, so that in an economic comparison based on 1925 tonnage it will be fair to use 80% of \$7 615 450, or \$6 092 000, as the measure of the annual charges in that year.

Tonnage in 1925.—Some of the papers in the Symposium use mileage and ton-mileage data for the calendar year 1923 and others for the calendar year 1924. As a study of these data and of the reports from which they were compiled disclosed certain discordant features, the method of collecting tonnage statistics on the Ohio for 1925 was changed, a higher standard of accuracy being established. This fact will undoubtedly account for the large increase in the quantities of sand and gravel; as reported for 1925, and may account for part of the other increases. In Table 22 comparison is made between the tonnage of 1923, 1924, and 1925, as reported, no attempt being made to correct any of the errors, plus or minus, that are believed to exist in the reports for 1923 and 1924.

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1645.

† *Loc. cit.*, November, 1925, Papers and Discussions, p. 1885.

‡ *Loc. cit.*, October, 1925, Papers and Discussions, p. 1667.

Mr. Alfred* deduced annual charges of 12.9 mills per ton-mile, but by using the revised annual charges and the tonnage of 1925 this rate becomes 7.25 mills per ton-mile.

Benefits of Savings Based on 1925 Tonnage.—While Mr. Alfred's annual charges on the waterway are regarded as too high, his water haulage cost, 2.7 mills per ton-mile, is too low. This rate does not differ greatly from the 2.5-mill rate set up by the writer for hauls of 250 miles; but it is not applicable to hauls of 86 miles (the average on the Ohio, exclusive of sand and gravel). Furthermore, his comparison of the sum of the two (annual charges per ton-mile plus water haulage cost per ton-mile) with the average railway revenue is unfair, as it fails to take into consideration the difference in the average length of haul, that on the railways being more than 200 miles whereas that on the Ohio is less than 90 miles.

TABLE 22.—VARIATION OF OHIO RIVER TONNAGE.

Commodities.	TRAFFIC FOR YEAR:		
	1923.	1924.	1925.
Coal and coke, in tons.....	6 041 708	5 811 552	6 881 354
Iron and steel, in tons.....	377 156	550 368	530 464
Oil and gasoline, in tons.....	50 187	82 200	275 532
Packet, in tons.....	277 073	290 029	263 393
Unclassified, in tons.....	332 471	455 657	998 521
Sand and gravel, in tons.....	1 201 925	3 746 882	6 768 585
Total, in tons.....	8 280 520	10 866 683	15 725 849
Total in ton-miles.....	708 802 795	670 486 423	845 000 000
Total in ton-miles, exclusive of sand and gravel...	693 865 289	621 391 597	757 000 000

The most direct method of showing benefits is by analyzing specific movements as was done by Mr. Begien in Table 17,† and that method will be used in connection with the analysis of the 1925 tonnage (Table 23). Some elements in Table 17, however, call for comment. The canalization project, Huntington, W. Va., to Cincinnati, Ohio, was not completed until late in 1925, but even under this handicap cost data covering operation in 1925 place the water haulage cost at 50 cents per ton as against 55 cents. It should be less hereafter. An elevating cost of 25 cents is fair for unloading plants of the old type. With modern plants on the Monongahela the cost is less than 10 cents. Making allowance for the smaller movement at Cincinnati, a cost of 15 cents is deemed to be fair. With these changes there is a saving of 68 cents per ton to river-side plants, and to householders within hauling radius of the river terminal 43 cents per ton on coal which requires a switching haul and 27 cents per ton on coal reshipped by rail to Indianapolis, Ind. An average saving on this movement is nearer 40 cents than 20 cents, as assumed by Mr. Begien.

* *Proceedings, Am. Soc. C. E.*, October, 1925, Papers and Discussions, p. 1667.

† *Loc. cit.*, November, 1925, Papers and Discussions, p. 1886.

TABLE 23.—SAVINGS ON OHIO RIVER COMMERCE FOR YEAR 1925.

Item No.	Commodity.	Origin.	Point of transfer to Ohio River.	Destination.	Average haul on tributary, in miles.	Average haul on Ohio, in miles.
(1)	(2)	(3)	(4)	(5)	(6)	
1	Coal	Monongahela River.	Pittsburgh, Pa.	Upper Ohio.	50	38
2	"	Logan Field.	Huntington, W. Va.	Cincinnati, Ohio, Louisville, Ky., and intermediate points.	Rail, 70	199
3	"	"	"	Portsmouth, Ohio.	Rail, 70	48
4	"	"	"	Indianapolis, Ind., etc.	Rail, 180	168
5	"	Kanawha River	Point Pleasant, W. Va.	Cincinnati, Ohio.	Rail, 60	204
6	"	West Kentucky Field.	Caseville, Ky.	Paducah, Ky., and Cairo, Ill.	Rail, 10	96
7	"	Monongahela River.	Pittsburgh, Pa.	Pittsburgh, Pa.	2
8	"	"	Miscellaneous Movements	100
9	Iron and steel	Monongahela and Ohio.	Pittsburgh, Pa.	Ohio and Monongahela.	8	59
10	"	Ohio River points.	Cairo, Ill.	Ohio River points.	178
11	"	Upper Ohio River.	Mississippi River.	Mississippi River.	487	926
12	Oil and gasoline	Parkersburg, W. Va.	Point Pleasant, W. Va.	Charleston.	58	81
13	"	Ravenna, Ky.	Carrollton, Ky.	Louisville, Ky.	223	63
14	"	Ohio River points.	"	Ohio River points.	"	150
15	Stone.	"	"	"	"	22
16	Cement.	"	"	"	"	63
17	Packet.	Ohio River and tributaries.	"	Ohio River and tributaries.	70	35
18	Logs and lumber	Ohio River points.	"	Ohio River points.	"	87
19	Unclassified	"	"	"	"	13
20	Sand and gravel.	"	"	"	"	54
Total or average.						

TABLE 23.—(Continued).

Item No.	Commodity.	Origin.	Point of transfer to Ohio River.	Destination.	Average haul on tributary, in miles.	Average haul on Ohio, in miles.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
21	Coal	Monongahela River	Pittsburgh, Pa.	Upper Ohio	50	38
22	"	Logan Field	Huntington, W. Va.	Cincinnati, Ohio, Louisville, Ky., and intermediate points	Rail, 70	199
23	"	"	"	Portsmouth, Ohio	Rail, 70	48
24	"	"	"	Indianapolis, Ind., etc.	Rail, 180	168
25	"	Kanawha River	Point Pleasant, W. Va.	Cincinnati, Ohio	Rail, 60	204
26	"	West Kentucky Field	Caseville, Ky.	Paducah, Ky., and Cairo, Ill.	Rail, 10	96
27	"	Monongahela River	Pittsburgh, Pa.	Pittsburgh, Pa.	2
28	"	"	Miscellaneous Movements	100
29	Iron and steel	Monongahela and Ohio	Pittsburgh, Pa.	Ohio and Monongahela	8	59
30	"	Ohio River points	Cairo, Ill.	Ohio River points	178
31	"	Upper Ohio River	Mississippi River	Mississippi River	487	926
32	Oil and gasoline	Parkersburg, W. Va.	Point Pleasant, W. Va.	Charleston	58	81
33	"	Ravenna, Ky.	Carrollton, Ky.	Louisville, Ky.	223	63
34	"	Ohio River points	"	Ohio River points	"	150
35	Stone	"	"	"	"	22
36	Cement	"	"	"	"	63
37	Packet	Ohio River and tributaries	"	Ohio River and tributaries	70	35
38	Logs and lumber	Ohio River points	"	Ohio River points	"	87
39	Unclassified	"	"	"	"	13
40	Sand and gravel	"	"	"	"	54
Total or average.						

TABLE 23.—(Continued).

Item No.	Average rail rate, per ton. (7)	Average cost, per ton, via water or rail and water. (8)	Terminal differential, per ton. (9)	Total savings, per ton. (10)	Tonnage. (11)	Savings accruing to tributaries. (12)	Savings accruing to Ohio river. (13)
1	\$1.34	\$0.30	Tipple, \$0.10* Unloading, 0.06	\$0.94	8 920 997	\$1 914 300	\$1 738 700
2	1.89	Rail, 0.50 Water, 0.60	Unloading, 0.15 Delivery, 0.125†	0.455	491 620	223 000
3	1.39	Rail, 0.50 Water, 0.25	Tipple, 0.06 Unloading, 0.15	0.43	874 002	161 000
4	2.53	Rail, 0.50 Water, 0.50	Tipple, 0.06 Unloading, 0.15	0.27	310 892	84 000
5	1.89	Rail, 0.125† Water, 0.90	Tipple, 0.06 Unloading, 0.875	0.305	646 755	45 000	183 000
6	1.35	Rail, 0.20 Water, 0.35	Tipple, 0.06 Unloading, 0.875	0.25	293 251	73 300
7	Delivery, 0.125†	480 110	Not estimated.
8	1.45	0.40	Tipple, 0.15 Unloading, 0.15	0.40§	354 817	142 000
9	1.18	0.54	Unloading, 0.15	0.34	273 816	11 000	82 000
10	3.81	1.09	Unloading, 0.25	1.75	165 675	290 000
11	11.50	5.65	Unloading, 0.75	4.85	99 973	160 000	395 000
12	3.87	1.47	Unloading, 0.40	2.00	275 532	133 000	418 000
13	0.60	0.20	Loading, 0.05 Unloading, 0.10	0.25	542 231	135 570
14	1.75	0.60	Unloading, 0.15	0.50	36 935	18 467
15	80% of rail rates.	Unloading, 0.15	1.00§	292 393	Not estimated.
16	80% of rail rates.	0.30§	186 415	124 250	62 140
17	0.30§	232 892	70 000
18	0.30†	6 708 585	1 353 717
19	Sixth class. 3.00
20	0.70
Total or average.	15 725 849	\$5 333 894

* River mines, river delivery, large movements, and efficient terminals. † Delivery cost of \$0.25 on one-half of tonnage. ‡ Rail cost of \$0.25 on one-half of tonnage. § Rough estimate without details of specific movements. ¶ Cost of dredging, transporting, and elevating sand and gravel is reduced \$0.20 per ton by canalization of the river. †† Traffic between steel mills having large tonnages and efficient terminals.

In connection with this rail-river-rail movement of coal to Indianapolis, the river is not regarded by the railroads as an integral part of the country's transportation system and is not accorded the same treatment as would be given a railroad; for coal brought to Cincinnati by river for shipment to Indianapolis pays a higher rate (Cincinnati to Indianapolis) than that brought in by rail, the differential being 18½ cents per ton.

Mr. Barnes' estimate of savings* is subject to a correction due to the fact that many of the Ohio River hauls which go to make up the average are parts of through hauls involving tributary streams and in some cases parts of through rail and river hauls. In such cases it is necessary to compare the through water rate or through rail-river rate with the through rail rate and apportion the savings between rivers. This may be illustrated by reference to the largest coal movement—Monongahela River points to the Upper Ohio. The total difference or saving averages 94 cents per ton on a total haul of 88 miles (50 miles on the Monongahela and 38 miles on the Ohio). The share accruing to the Ohio is 44 cents, or 11.5 mills per ton-mile.

It has been alleged† that the economic comparisons are faulty in that they fail to include in the cost of transportation by river either the terminal cost of delivery or any profit to the carrier. In Table 23 based on the tonnage of 1925, as well as in all other comparisons made by the writer, a terminal differential has been included. As to whether profit was included depends on the type of service that is, whether by private carrier, contract carrier, or common carrier. In dealing with private carrier movements cost only was considered, the cost including a 6% return on the investment in floating plant and on the privately owned facilities. Although such a return may not be regarded as embodying a profit, it is equal to or in excess of what the railroads are permitted to earn on their investment under the Transportation Act. In dealing with contract carrier or common carrier movements, rates have been used which do include an item of profit.

In preparing Table 23, cost data covering a large number of movements were studied, but in order not to disclose costs of individual movements received confidentially, they have been consolidated into groups and the rates and savings listed are the average for the group.

The saving on sand and gravel is more or less indeterminate. While a series of comparisons indicate an average saving of \$0.50 per ton, only part of this amount can be fairly credited as a benefit resulting from the canalization project as the industry would exist without the dams. On the other hand the dams in many cases double the length of the season and give access to deposits that could not otherwise be profitably mined. The saving of \$0.20 per ton is the estimated reduction in the cost of producing sand and gravel.

It will be noted from Table 23 that the accumulated savings for 1925 amount to 87% of the annual charges for that year and to 70% of the estimated annual charges on completion.

* *Proceedings, Am. Soc. C. E.*, January, 1926, Papers and Discussions, p. 138.

† *Loc. cit.*, p. 136.

Prospective Tonnage.—If Mr. Alfred* is right in his assumption that the commerce of the Ohio will not increase but rather decrease, his conclusion is justified although his reasons are not convincing. It is true that the per capita production of coal is decreasing—and that the actual production, with minor fluctuations, has remained stationary since 1913, but the quantity moving on the Ohio has been increasing since 1920 when the movement from Pittsburgh to points on the Mississippi River ceased.

It is true that coal will move increasingly to central power stations as a substitute for coal formerly moved to local stations, but as such stations require 500 tons of condensing water for each ton of coal consumed they must locate on large rivers. Furthermore, if they can secure their coal by water, it is more economical to build such stations near the center of distribution than at the mouths of coal mines 200 miles distant.

It is true that petroleum is taking the place of coal, but the prospect that the Ohio River will receive a large share of this substitute tonnage is shown by the remarkable increase in this item in 1924-25.

While in all probability coal will always remain the largest single item of tonnage, the prospective movement of steel and iron to points on the Mississippi River bids fair to yield the largest returns eventually. Although a seasonal movement is all that is possible at present, the large steel companies in the Ohio Valley foreseeing the approaching completion of the Ohio improvement, already have taken steps to move their products by water on a large scale. In addition to seasonal movements to the Lower Mississippi, a common carrier service has been established whereby such products are being shipped by river to Louisville, thence by rail to St. Louis, Mo., and Memphis, Tenn., pending completion of the canalization project.

Mr. Begient† makes the point that "the comparatively few industries and individuals who make use of river transportation contribute a very small part toward the interest and carrying charges of such improvements" and that in connection with the movement of coal by river none of the saving in transportation costs is passed on to the consumer.

The theory underlying the improvement of rivers at public expense assumes that the savings in transportation costs will be transmitted to the taxpayers who pay the annual charges, or to a large and representative group thereof. Not all the tax-paying public is benefited by a given improvement, but it may benefit to an equal extent from some other similar public improvement. If, however, the saving is not passed on as alleged then some means should be devised for placing the maintenance burden on the users. This could be done either through the collection of tolls or by a licensing system similar to that which governs the use of highways created at public expense, as was suggested a number of years ago by a prominent waterways advocate.

This question, however, is separate and distinct from the one at issue, namely, whether a canalized Ohio River will serve as an economic link in the country's transportation system. The writer believes that the use now being

* *Proceedings, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1669.*

† *Loc. cit., November, 1925, Papers and Discussions, p. 1887.*

made of the river, the savings now being effected, and the prospective increase in use on completion of the project three years hence justify an affirmative answer.

The Ohio River's full usefulness, however, will not be realized until the Interstate Commerce Commission is given the same control over rates by river as it now has over rail rates, and until the railroads recognize the value of the river by granting joint rail and river rates as freely as they grant joint rail rates.

It is true that coal will move increasingly to central power stations situated far from the river. Furthermore, if they can secure their coal by water, it is more economical to build such stations near the coast of distant than at the mouth of coal mines 800 miles distant.

It is true that petroleum is taking the place of coal, but the prospect that Ohio River will receive a large share of this substitute tonnage is shown by the remarkable increase in this item in 1924-25.

While in all probability coal will always remain the largest single item of cargo, the prospective movement of steel and iron to points on the Mississippi river this far to yield the largest returns eventually.

Although a seasonal movement is all that is possible at present, the large steel companies in the Ohio River forecast the approaching completion of the Ohio improvement.

They have taken steps to move their products by water on a large scale. In addition to seasonal movements to the lower Mississippi, a common car service has been established whereby such products are being shipped by rail to Louisville, thence by rail to St. Louis, Mo., and Memphis, Tenn.

which completion of the canalization project.

Mr. Hooton makes the point that "the comparatively few industries and individuals who make use of river transportation contribute a very small part toward the interest and carrying charges of such improvements," and that in question with the movement of coal by river none of the saving in transportation costs is passed on to the consumer.

The theory underlying the improvement of rivers at public expense assumes that the savings in transportation costs will be transmitted to the taxpayers to pay the annual charges or to a large and representative group thereof.

At all the tax-paying public is benefited by a given improvement, but it may not to an equal extent from some other similar public improvement. It is argued that the saving is not passed on as alleged, then some means should be found for placing the maintenance burden on the users.

This could be done either through the collection of tolls or by a licensing system similar to that which governs the use of highways created at public expense, as was suggested a number of years ago by a prominent waterways advocate.

This question, however, is separate and distinct from the one at issue, namely, whether a canalized Ohio River will serve as an economic link in the nation's transportation system. The writer believes that the use now being

THE HEXAGONAL SLAB DESIGN OF CONCRETE PAVEMENT

Discussion*

By MESSRS. H. O. ROOT AND WESLEY VANDERCOOK

H. O. Root,† Esq. (by letter).‡—The theory of the hexagonal design of concrete pavement is simple and has been ably handled in this original paper. There may be, however, some question in the minds of many engineers as to its practicability. The writer, having had two years of field experience in laying the hexagonal design, feels that a detailed description of field methods would be a fitting contribution to this discussion.

Narrow Streets and Highways.—When the street or highway is of such width that it can be screeded successfully from edge to edge and the design is a single staggered center joint, the placing of the interior joints is very simple.

The start is a matter of special plan in each case, and, if made from an existing straight pavement edge, will result in one smaller and one larger slab, the end of the first length of center joint being brought to a point off center the required distance at the end of the first transverse joint.

Assume, for illustration, that the pavement is to be 24 ft. wide and that the design calls for a single staggered center joint of 10-ft. lengths. These will form 120° angles with intersections 2.5 ft. off center, and the transverse joints from these points to the edge will be 9.5 ft. in length.

The contractor provides metal "saws", or strips, in both the 9.5 and 10-ft. lengths. One end of a 10-ft. saw is secured to the first fixed angle point established, its free end is brought to an intersection with a 9.5-ft. saw and the latter adjusted to be at right angles with the edge. The location of each 10-ft. center length is, therefore, automatically established by bringing it to meet the end of the next 9.5-ft. transverse joint. The transverse joints can be kept square with the edge by eye and should fall at intervals of 17.32 ft. on each edge.

Intersections of streets must be carefully planned in advance, and it may be necessary to shorten or lengthen two or three panels at the end of a block so as to join in correctly with the intersection plan. Thin boards are used instead of metal saws where odd lengths are required. When panel plans of both block and intersection are supplied to the field engineer and the inspector, there need be no delay to the mixer while constructing irregular layouts.

* Discussion of the paper by Lewis A. Perry, Assoc. M. Am. Soc. C. E., continued from March, 1926, *Proceedings*.

† Los Angeles, Calif.

‡ Received by the Secretary, February 16, 1926.

The fact that the intersection of the center joint with the transverse joint falls off center gives the joint setter more freedom of action, for he is not required to work directly under the boom of the mixer, and the to-and-fro movement of the bucket does not interrupt him, nor does his work in any way interfere with the rapid deposit of concrete by the mixer operator.

Experience proved that it was more efficient to move the mixer and pull up the power sub-grade drag only far enough to allow one length of center joint and one transverse joint to be set at a time. These short moves can be made without loss of mixing time and tend to simplify the organization of the work on the sub-grade and joints between the mixer and the line of deposit.

Actual experience covering two seasons' work and a yardage of more than 400 000 seems to warrant this assertion, that the hexagonal design actually makes for higher efficiency and, therefore, for better progress and lower costs than is usually attained with the standard rectangular design.

Wide Streets.—When the improvement is of such width that it cannot be laid completely at one trip, it is necessary to set one straight form outside the line of hexagon points to be laid in the first operation. The sub-grade may then be brought to correct crown and grade by a template drag riding on this false form and the curb, or edge, form.

The contractor provides a supply of 3-in. form lumber, of a width equal to the thickness of the pavement. These pieces are cut so that the ends have a bevel of 60°, the two bevel cuts being parallel, and the face length equal to the designed length of the joints. The straight false form is marked off in panel-length intervals and the extreme ends of two beveled pieces are brought to these marks; their adjacent ends are then brought together on the grade and secured by staking. The setting of these forms in pairs to the marks on the false form is a simple operation and increases only slightly the whole cost of form-setting.

The concrete is then struck off, tamped, and finished, all from the false form; the beveled forms are merely trued up to the rod as it passes over them. Longitudinal expansion joint material is placed with the first strip, flush with the form, and secured to the concrete by small nails driven through flat tin flashers, adequately spaced and located near the top of the pavement. This holds the material firmly against the concrete and prevents loosening and flopping after the forms are removed.

A long straight-edge should be used on the projecting points of concrete during the finishing process. It is important that these areas be reasonably true as to surface contour, for they must fit the crown of the tamping rod as it idles over them during the laying of the adjacent strip.

The beveled forms may be removed and used elsewhere the following day, but the false form should be left in place. The remainder of the sub-grade, or such part of it as will be required for the next run, may be brought to its true crown by means of the template drag sliding on this form. The open triangles between the form and the old concrete must be smoothed up by

hand. After the sub-grade has been struck off and before the concrete is placed, this false form should be removed and all tamping and finishing done from the hardened surface of the old concrete.

Here, again, it is important that the long straight-edge be used carefully where the triangles of new concrete mesh in with the old. A series of humps or hollows, even if slight, occurring as they do at regular intervals, will seriously affect the riding qualities of the pavement. As the difference in cost between a smooth finish and a rough one is probably less than 1 cent per sq. yd., it is the writer's opinion that the public would gladly pay this difference and secure the better riding quality.

Wide Areas.—Alternate tiers of hexagons may be laid between false forms as described previously, the filler tiers being screeded and finished to the surface of the old tiers, with no forms other than division boards.

General.—Complete separation of the slabs from one another is important. Careful edging and complete separation will prevent spalling and the subsequent raveling at corners and along joints. The longitudinal joint material may be run continuous through the three-way intersection and all splices made to occur at least 2 ft. from the angle. The concrete deposited in the angle of the continuous joint tends to keep it butted firmly against the end of the transverse joint. It is possible that some metal clip device could be used to hold the end of the transverse joint in true position.

Where manholes occurred on or near the line of joints, the joints were broken and turned to intersect them squarely. An extra thickness of concrete was put in to compensate for any section reduction caused by manholes, inlets, or other features.

An instrument crew can be kept steadily employed with each paving mixer, staking and checking forms, laying out intersections, and checking crowns on both sub-grade and finished concrete. Close co-ordination in this respect is reflected in the progress, quality, appearance, and cost of the finished work.

Conclusions.—The laying of hexagonal slabs is not as difficult as appears at first glance. The radical change in design is not confusing to workmen and even ceases to be a novelty after the first few panels have been laid. The design permits of a highly efficient organization of the labor, which, in turn, results in rapid progress and low costs for the contractor. The item of increased cost soon disappears under the pressure of competitive bidding.

The methods here detailed will no doubt be improved by other engineers and contractors when the use of the hexagonal design becomes more general. It is with this thought in mind that the writer presents this discussion, more as a basis for further refinement in practice than as a set of hard and fast rules.

WESLEY VANDERCOOK,* Esq. (by letter).†—The fact that such widely different views are held concerning the essentials of concrete pavement design

* Chf. Engr., The Long-Bell Lumber Co., Longview, Wash.

† Received by the Secretary, February 23, 1926.

should promote research of the character reported in Mr. Perry's paper. Even if no definite refinement of design resulted from this work, the subsequent investigations on a larger scale (which, unfortunately, are not reported in the paper) and the better acquaintance with the causes and character of typical failures in pavement slabs have completely justified the effort made.

It seems necessary to the intelligent discussion of the paper to note that the designer of this type of pavement is among those who believe in segregating a pavement into certain definite slab units and eliminating cracking by meeting Nature "half way". To this school of thought the paper should be of interest, if not of value. There still are, however, adherents to the custom of constructing pavement in slabs of full width and in excessive or indefinite lengths, who rely on Nature to use her best judgment about cutting them up into smaller units.

Serious consideration of the paper pre-supposes that the former group of designers is thinking rightly, which leads to the question of slab sizes. Since the structural effect is of prime importance, probably the soundest argument for moderate slab sizes is the fact that the expansion of over-sized slabs, with insufficient expansion provision, sets up compressive stresses which express themselves in transverse flexure. This becomes harmful when sympathetic transverse loading is applied and probably accounts for much pavement breakage. Random transverse cracks often follow local lines of least resistance and frequently terminate at the edge with an acute corner, which suffers under cantilever loading even more heavily than the right-angled corner. In the case of city streets, of course, this is not as serious as on highways, where the traffic is concentrated along the edge.

Probably the best argument in favor of the relatively long slab is the apparent saving in first cost of expansion joint material. The writer, however, questions whether within a few years, this saving, and more, has not been lost in the "hot pot" by the maintenance crew. Certainly the appearance of pavements, especially of city streets, is highly important, even if secondary to utility, and, in the writer's mind, cracks in a pavement, whether raw and raveled or treated with asphalt, are decidedly unsightly. It is unfortunate that the logical asphalt treatment serves to accentuate these flaws.

The writer, therefore, endorses the practice of segregating pavements into slabs, sized as nearly as may be to the ideal unit. Probably the size will vary with conditions, in fact, all the conditions having a bearing on the correct slab size may not be known as yet. Some of these conditions are range of temperature, character of subsoil, width of roadway, position of loading concentrations (if defined), and the correlated consideration of slab shape, which the paper discusses.

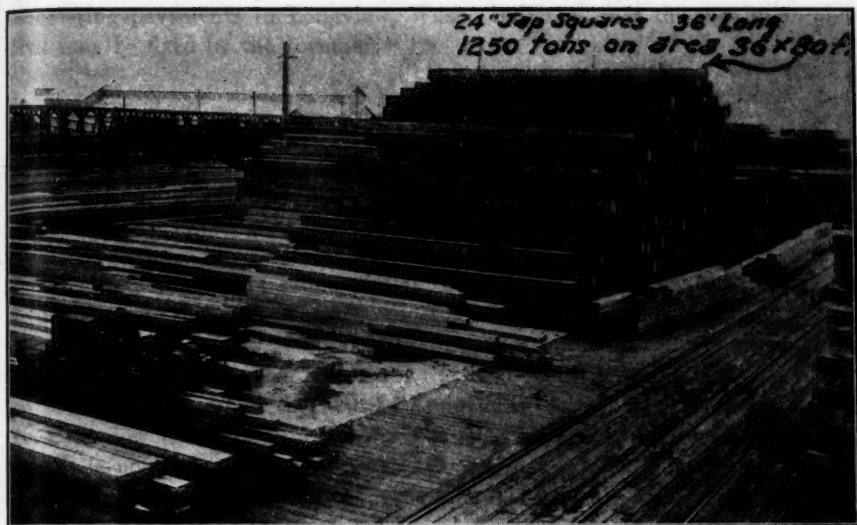


FIG. 12.—VIEW OF HEAVY STORAGE OF LUMBER ON HEXAGONAL SLAB PAVEMENT, LONG-BELL LUMBER CO., LONGVIEW, WASH.



FIG. 13.—VIEW OF LOADING AND TRAFFIC ON HEXAGONAL SLAB PAVEMENT. SAME CONSTRUCTION AS SHOWN ON AUTHOR'S FIG. 5 AND IN FIG. 12.



FIG. 1. PERSPECTIVE VIEW OF THE NEWLY CONSTRUCTED CONCRETE PAVEMENT, LONG BEACH, CALIF., SHOWING THE EFFECT OF THE JOINTS.



FIG. 2. PERSPECTIVE VIEW OF THE NEWLY CONSTRUCTED CONCRETE PAVEMENT, LONG BEACH, CALIF., SHOWING THE EFFECT OF THE JOINTS.

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Of these conditions perhaps the most important is temperature. A survey of existing pavements in Longview, Wash., indicates that the maximum slab area may be fixed by the formula:

$$A = d y$$

in which,

A = the area of the slab, in square feet;

d = the depth of the pavement, in inches; and

$$y = 100 - \left(\frac{\text{temperature range, in degrees Fahrenheit}}{2} \right)$$

For a local temperature range of 100° this would set the maximum size of a 7-in. pavement slab at 350 sq. ft., and for a range of 140° at 210 sq. ft. Sub-grade conditions in any given locality, of course, enter into this determination. The foregoing values seem logical for the local conditions, and this rule should be followed only when the slab is compact in shape, preferably hexagonal. In Longview the minimum slab area is fixed by a minimum weight of 7 000 lb. It is not to be expected that these empirical rules will hold good for any or all localities, but they may suggest a logical measure of safe dimensioning whereby slabs may be intelligently designed rather than merely jointed according to some custom that may have been established under dissimilar conditions.

Mr. Hooper* probably would not have considered his objections to this type applicable or important had he had an opportunity to watch closely its development, and the apparently highly successful performance of the resulting improvement. His comment seems to be directed chiefly at this design as applied to narrow street or highway pavements, as shown in Fig. 6,† and has apparently missed the importance of the increased transverse section at mid-slab. Mr. Hooper does not comment on the advantages of this principle for a street three, four, or five panels in width. Certainly, the wider the street the greater will be the number of corners and the greater the value of the design.

The author's Fig. 5† was taken immediately after construction; Figs. 12 and 13 show the normal heavy traffic and storage on this pavement. It is 6 in. thick and was built on a hydraulic sand fill of from 6 to 35 ft. in depth. The slabs are slightly in excess of the allowable area as determined by the formula, being 314 sq. ft. Since this improvement shows no signs of structural defect after twenty months of this usage, it is considered reasonable proof of the fitness of the design. The writer doubts whether any other type of pavement at the same, or nearly the same, cost would have proven as satisfactory.

The author compares the 6-in. hexagonal corner with the 7-in. rectangular corner, which no doubt is correct reasoning. The writer, however, cannot agree with the implication that it is good practice to minimize the thickness of pavement of hexagonal design in the interest of first cost economy. It is thought that, except in rare cases where maximum loading is fixed for all

* *Proceedings, Am. Soc. C. E.*, January, 1926, Papers and Discussions, p. 158.

† *Loc. cit.*, November, 1925, Papers and Discussions, p. 1801.

time, it is better practice to maintain depths heretofore considered standard and enjoy the increased value of the improvement by reason of the design, rather than to hold to capacities of existing standards at reduced cost. In this connection it may be remembered that the structural value of the slab itself increases with the square of the depth, whereas the cost does not quite vary directly with the depth.

The hexagonal design has passed the experimental stage, has proved to have structural merit peculiar to its shape, and is of considerable economic value. The 120° corner has been found approximately 50% stronger than the 90° corner under cantilever loading on sub-grades. It is doubtful if this value can be ignored by engineers. While it is questionable whether this type will find immediate acceptance by designers, because conservatism in the engineer is a virtue, the writer ventures the prediction that this principle will find favor when it has been investigated in theory and practice as thoroughly as Mr. Perry has done.

The author's Fig. 5† was taken immediately after construction; Figs. 12 and 13 show the normal heavy traffic and storage on this pavement. It is 12 in. thick and was built on a hydraulic sand fill of from 6 to 35 ft. in depth. The slabs are slightly in excess of the allowable area as determined by the formula, being 314 sq. ft. Since this improvement shows no signs of structural defect after twenty months of this usage, it is considered responsible proof of the fitness of the design. The writer doubts whether any other type of pavement at the same, or nearly the same, cost would have proven as satisfactory.

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† Proceedings, Am. Soc. C. E. January, 1926, Papers and Discussions, p. 158.
 ‡ Loc. cit., November, 1925, Papers and Discussions, p. 1801.

STRESSES IN HELICALLY REINFORCED CONCRETE COLUMNS

Discussion*

By MISSAC THOMPSON, Assoc. M. Am. Soc. C. E.

MISSAC THOMPSON,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper with its tabulation of experimental results, is certainly of interest to all engineers concerned with the design of concrete columns. It would have been far more interesting from a designer's point of view, if some data were included as to measurements of the actual shortening of columns under working loads with and without helical reinforcement. There has never been any real question in regard to the effectiveness of helical reinforcement as far as the ultimate strength of the column is concerned under the action of direct load, but rather as to what extent, if any, the helical reinforcement strengthens the column when the unit pressure on the concrete is 60% or less of its crushing strength.

Generally speaking, the total direct load coming on a column will seldom be greater than the amount a properly designed column can carry. A factor of safety is chiefly required to provide against (1) poor concrete; and (2) flexure due to unbalanced loading, rigidity, unhomogeneity, etc.

It is obvious that the latent strength of helically reinforced concrete columns will serve to good advantage when the concrete is poor, but that against flexure it will be of little use. It is a matter of judgment for a designer to decide when to rely on helical reinforcement. In one and two-story building columns the effect of flexure is important and, therefore, helical reinforcement is out of place. In a multiple-story building, however, the effect of poor concrete is more important, hence helical reinforcement serves a useful purpose.

In a reinforced concrete structure the amount of flexure to which a column is subjected, is indeterminate. The writer's observations on the behavior of concrete columns makes him skeptical about results based on the rigid frame theory. The construction joints materially change the results. The actual flexure on interior as well as exterior columns is invariably much less than that given by the rigid frame theory. Moreover, shrinkage, unequal settlement, temperature change, etc., will develop flexure in columns that could not be computed by any reasonable theory. In columns the reserve strength against flexure as well as against direct load is the only basis for proper design

* This discussion (of the paper by A. W. Zesiger, M. Am. Soc. C. E., and E. J. Affeldt, Assoc. M. Am. Soc. C. E., published in January, 1926, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Engr. and Builder, Brooklyn, N. Y.

‡ Received by the Secretary, February 8, 1926.

within the limits of the same factor of safety as is used for other parts of the structure.

It is to be noted that even if the pressure on the cross-section of the column is not uniform, as long as there is no reversal of stress on any part of the column, the resulting lateral elongation (and, consequently, the stress in the spirals and the resulting resistance to shearing failure) will be the same as if the pressure was uniformly distributed over the cross-section. Therefore, in that case, helical reinforcement will be useful to a certain extent in providing against flexure.

In the case of direct load and flexure such as will not cause any reversal of stress in any part of the column, the author's reasoning regarding the benefit of helical reinforcement still holds, except that V of Equation (21)* must be reduced by the ratio of average to maximum stress.

The authors have disregarded the fact that, as the column shortens, each spiral coil lengthens by an amount:

$$\frac{e_s}{\text{Number of coils per unit of height}}$$

It is evident that, for columns 20 in. or more in diameter, this omission is of no consequence, but for smaller columns it affects the results considerably.

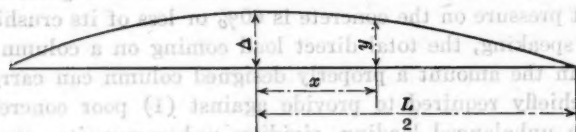


FIG. 7.

General Theory.—A general review from a theoretical standpoint of a homogeneous column under axial load is of interest. Consider a homogeneous column with hinged ends loaded axially:

Let A = cross-section of the column;

P = total load on the column;

I = least moment of inertia of the column section;

r = radius of gyration corresponding to I ;

L = length of the column;

E = modulus of elasticity;

C = perpendicular distance from the extreme fiber to the neutral axis on which I is taken;

Δ = maximum deflection of the column under axial load;

R = radius of curvature of the bent column at any point, X, Y ;

M = moment at any point of the column;

U = extreme fiber stress;

K = factor of safety.

Referring to Fig. 7, the column when bent, is subject to a positive moment,

$$M = P Y \dots \dots \dots (32)$$

Assuming Y to be positive upward,

$$\frac{EI}{Py} = \frac{-\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{\frac{3}{2}}}{\frac{d^2y}{dx^2}} \dots (33)$$

Assume the curve to be so flat that $\frac{dy}{dx} = 0$, Equation (33) will reduce to:

$$-\frac{EI}{Py} = \frac{d^2y}{dx^2} \dots (34)$$

or,

$$-\frac{Py}{EI} dy = \frac{d^2y}{dx^2} \cdot \frac{dy}{dx} dx$$

Integrating,

$$-\frac{Py^2}{EI} = \left(\frac{dy}{dx}\right)^2 + \text{constant} \dots (35)$$

When $X = 0$, $Y = \Delta$, $\frac{dy}{dx} = 0$, and, therefore,

$$-\frac{Py^2}{EI} = \left(\frac{dy}{dx}\right)^2 - \frac{P\Delta^2}{EI}, \text{ or } \left(\frac{dy}{dx}\right)^2 = \frac{P}{EI} (\Delta^2 - y^2) \dots (36)$$

$$\frac{dy}{\sqrt{\Delta^2 - y^2}} = dx \sqrt{\frac{P}{EI}}$$

Integrating,

$$\sin^{-1} \frac{y}{\Delta} = x \sqrt{\frac{P}{EI}} + \text{constant} \dots (37)$$

Since, when $X = 0$, $Y = \Delta$ (constant) $= \sin^{-1} 1$:

$$\sin^{-1} \frac{y}{\Delta} = x \sqrt{\frac{P}{EI}} + \sin^{-1} 1 \dots (38)$$

Equation (38) represents the relation between X and Y in transcendental form.

The maximum stress in the column,

$$U = \frac{P}{A} + \frac{P\Delta}{S} \dots (39)$$

in which, $S = \frac{Ar^2}{c}$

Giving P in terms of the maximum deflection and maximum stress in Equation (38):

$$\sin^{-1} \frac{y}{\Delta} = x \sqrt{\frac{uAr^2}{(r^2 + \Delta c)EI}} + \frac{\pi}{2} \dots (40)$$

When $X = \frac{L}{2}$, $Y = \Delta$, and from Equation (40):

$$\Delta = \frac{uL^2 - \pi^2 EI r^2}{\pi^2 E c} \dots (41)$$

From Equation (41) it is evident that, when $U = \frac{\pi^2 E r^2}{L^2}$, the maximum deflection at the center of the column = 0, and that when $u > \frac{\pi^2 E r^2}{L^2}$, there is actual deflection at the center; for values of $u < \frac{\pi^2 E r^2}{L^2}$, the deflection is negative and therefore imaginary.

Substituting the value of Δ from Equation (41) in Equation (39),

$$\frac{P}{A} = \pi^2 E \left(\frac{r}{L} \right)^2 \dots \dots \dots (42)$$

Summary of Results.—

1.—Axially loaded homogeneous columns do not bend as long as $\frac{P}{A} < \pi^2 E \left(\frac{r}{L} \right)^2$. For this condition, therefore, the average stress, $\frac{P}{A}$, equals the maximum stress, U .

2.—When the average stress, $\frac{P}{A} = \pi^2 E \left(\frac{r}{L} \right)^2$, the column begins to buckle and the maximum stress increases and continues to increase with the deflection without any change in the average stress. In other words, once a column buckles it will continue to buckle without increase in the load; therefore, it is not a stable structure. The proper conclusion is that for stability $\frac{P}{A}$ must be less than $\frac{\pi^2 E}{K} \left(\frac{r}{L} \right)^2$, in which, K , is a proper factor of safety.

Taking 4 as the proper value for K , 2 000 000 as the proper value for E , and 500 as the proper value for $\frac{P}{A}$, the ratio of the length to the diameter of a round column must not exceed 25; and the ratio of the length to the side of a square column must not exceed 29. Within these ratios of length to diameter or side, the crushing value of concrete or its corresponding shearing resistance is the criterion for the strength of a column.

The maximum stress in the column, U , is given by Equation (41) as follows:

$$U = \frac{P}{A} + \frac{\pi^2 E r^2}{L^2} \left(\frac{1}{2} - \frac{1}{2} \cos \frac{\pi x}{L} \right) \dots \dots \dots (43)$$

which, at the center of the column, $x = \frac{L}{2}$, becomes

$$U = \frac{P}{A} + \frac{\pi^2 E r^2}{L^2} \left(\frac{1}{2} - \frac{1}{2} \cos \pi \right) = \frac{P}{A} + \frac{\pi^2 E r^2}{L^2} \dots \dots \dots (44)$$

When $U = \frac{P}{A}$, $\cos \pi = 1$, and from Equation (41) it follows that

$$\frac{P}{A} = \pi^2 E \left(\frac{r}{L} \right)^2 \dots \dots \dots (45)$$

which is the same as Equation (42).

Therefore, when the average stress equals the maximum stress, the column is at the point of buckling.

EVAPORATION ON UNITED STATES RECLAMATION PROJECTS

Discussion*

BY MESSRS. H. S. KLEINSCHMIDT, ROBERT FOLLANSBEE, R. I. MEEKER, AND
B. E. TORPEN

H. S. KLEINSCHMIDT,† M. Am. Soc. C. E. (by letter).‡—As indicated by the author, the question of evaporation is of great importance to those interested in reservoirs in the western part of the United States. There has always been considerable uncertainty and difference of opinion on this subject, and the investigation to be made by the Society's Special Committee on Irrigation Hydraulics will be welcomed. It is to be hoped that this will be undertaken on broad lines.

The author is to be complimented on having assembled his information in such good form. As stated by him, the obtaining of dependable information has been attended by many circumstances affecting the reliability of the records. To one not familiar with this subject it might seem that it should be a simple matter but it is far from simple.

No mention is made of precipitation. The writer believes that definite information should be furnished in this connection. The question at once arises whether the data submitted are for actual total evaporation, or for total evaporation less precipitation. This question has frequently arisen in the writer's experience, some engineers being in the habit of speaking of evaporation with precipitation deducted, and others without taking it into account.

Also, information is desirable as to method of handling pans, that is, whether water is added daily, weekly, monthly, etc., to keep the water level up to a fixed mark. As the keeping of records for a floating pan is much more difficult than for a land pan the comparative values between floating and land pans are especially valuable.

ROBERT FOLLANSBEE,§ M. Am. Soc. C. E. (by letter).||—This paper presents a mass of valuable information on a subject that is of especial importance to hydraulic engineers in the Western States, and in regard to which existing literature is meager. The measured evaporation from ice is the only record of that kind the writer has seen.

* This discussion (of the paper by Ivan E. Houk, M. Am. Soc. C. E., published in January, 1926, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Salt Lake City, Utah.

‡ Received by the Secretary, January 25, 1926.

§ Dist. Engr., U. S. Geological Survey, Denver, Colo.

|| Received by the Secretary, February 5, 1926.

The records as the author has given them represent evaporation measured in pans of different sizes, either set in the ground, resting on top of it, or floating in ponds and reservoirs. In the form given, the records are not all comparable, nor do any of them represent directly the evaporation from reservoir surfaces, the phase of evaporation that chiefly concerns the hydraulic engineer.

TABLE 5.—FACTORS TO REDUCE OBSERVED EVAPORATION
TO RESERVOIR EQUIVALENTS.

Size of pan.	Reduction factor.
Class A Station, 4 ft., circular, 10 in. deep, on ground.....	0.86
3 by 3 ft. by 18 in., floating.....	0.91
3 by 3 by 3 ft., set in ground.....	0.80
6 ft., circular, 2 ft. deep, set in ground.....	0.90
Class A Station, floating.....	0.92
3½ ft. circular, 22 in. deep, floating.....	0.90

Fortunately, data are available for the reduction of these records to what have been called "reservoir equivalents" or the corresponding evaporation from the surface of a large body of water, under the same conditions of temperature, relative humidity, and wind velocity. The most comprehensive experiments to determine the proper reduction factors for pans of various diameter, depth, and immediate surroundings were those made by the Office of Public Roads and Rural Engineering in Denver, Colo., from November, 1915, to November,

TABLE 6.—MEAN MONTHLY EVAPORATION, IN INCHES, REDUCED
TO RESERVOIR EQUIVALENTS.

Month.	Mesilla Park.*	Elephant Butte.	Roosevelt.	Yuma, citrus.	Yuma, evaporation.	Provo.
January.....	1.98	1.94	1.54	2.89	2.14	(0.68)
February.....	2.88	2.92	2.17	4.01	2.80	(0.77)
March.....	5.10	5.30	3.57	5.51	3.99	(2.40)
April.....	6.37	7.12	5.08	7.19	5.10	2.84
May.....	7.48	8.91	7.14	9.45	5.70	3.74
June.....	7.95	9.30	8.56	10.63	6.20	4.16
July.....	7.42	8.08	8.29	11.76	7.20	4.46
August.....	6.49	7.13	7.05	10.28	7.10	3.81
September.....	5.20	6.11	5.80	8.00	5.57	2.74
October.....	4.04	5.24	3.88	5.53	3.77	1.61
November.....	2.52	2.67	2.44	3.58	2.41	0.83
December.....	1.70	2.02	1.49	2.45	1.83	(0.60)
Total.....	59.13	66.74	57.01	81.28	53.81	28.64

* Called "Agricultural College" in Weather Bureau reports.

1916.* The comparison between evaporation from a Class A pan and a 12-ft. pan has been checked by experiments made by the U. S. Geological Survey in Escalante Valley, near Milford, Utah, during 1925.

The factors of Table 5, applicable to most of the author's records, have either been taken directly, or deduced, from the Denver experiments.

* *Journal of Agricultural Research*, Vol. 10, July 30, 1917, pp. 209-242.

TABLE 7.—MEAN MONTHLY RESERVOIR EQUIVALENT FOR EVAPORATION STATIONS IN AND ADJACENT TO COLORADO RIVER BASIN.*

Month.	Reservoir equivalent, in inches.	Temperature, in degrees Fahrenheit.	Wind velocity, in miles per hour.	Month.	Reservoir equivalent, in inches.	Temperature, in degrees Fahrenheit.	Wind velocity, in miles per hour.
WAGONWHEEL GAP, COLO., 1920-1924.				SANTA FE, N. MEX., 1913-1914; 1916-1925.			
Jan.....	(0.85)	15	1.7	Jan.....	1.13	29	2.8
Feb.....	(0.77)	18	1.8	Feb.....	1.50	34	3.1
Mar.....	(1.21)	22	2.2	Mar.....	2.75	38	3.6
Apr.....	(1.95)	29	2.4	Apr.....	4.27	46	3.9
May.....	(2.83)	42	2.6	May.....	5.94	56	3.5
June.....	3.36	53	2.6	June.....	7.01	66	3.0
July.....	3.04	55	2.2	July.....	5.80	69	2.1
Aug.....	2.36	52	1.8	Aug.....	5.14	67	1.6
Sept.....	2.10	46	1.7	Sept.....	4.48	61	1.8
Oct.....	1.35	35	1.7	Oct.....	3.85	50	2.2
Nov.....	(1.17)	24	1.5	Nov.....	1.98	40	2.4
Dec.....	(0.78)	17	1.4	Dec.....	1.07	30	2.5
Year.....	21.77	34	2.0	Year.....	44.42	49	2.7
MYTON, UTAH, 1918-1925.				FARMINGTON, N. MEX., 1915-1925.			
Jan.....	(0.43)	14	(3.1)	Jan.....	1.69	38
Feb.....	(0.65)	23	(3.6)	Feb.....	1.33	41
Mar.....	(1.68)	36	4.3	Mar.....	2.88	48
Apr.....	4.14	46	4.1	Apr.....	4.90	54
May.....	5.92	57	3.4	May.....	6.34	63
June.....	6.75	66	2.9	June.....	6.90	71
July.....	6.50	72	2.4	July.....	7.24	74
Aug.....	5.55	70	2.4	Aug.....	5.62	72
Sept.....	4.15	61	2.6	Sept.....	4.87	64
Oct.....	2.48	48	2.5	Oct.....	3.20	54
Nov.....	(1.20)	34	2.1	Nov.....	1.76	44
Dec.....	(0.40)	20	(1.6)	Dec.....	.87	38
Year.....	39.85	46	3.0	Year.....	46.10	55
PIUTE DAM, UTAH, 1918-1925.				WILLCOX, ARIZ., 1917-1925.			
Jan.....	(0.94)	26	(3.4)	Jan.....	2.25	41	3.8
Feb.....	(1.08)	31	(3.5)	Feb.....	3.11	44	3.9
Mar.....	(1.79)	37	(3.8)	Mar.....	5.12	48	4.4
Apr.....	4.84	44	3.9	Apr.....	6.67	55	4.5
May.....	6.08	56	3.6	May.....	7.78	63	3.5
June.....	7.81	64	3.4	June.....	8.06	72	2.8
July.....	7.11	71	2.8	July.....	7.02	76	2.5
Aug.....	6.81	68	2.8	Aug.....	5.95	73	1.8
Sept.....	5.20	60	2.9	Sept.....	5.35	69	2.3
Oct.....	3.35	47	3.0	Oct.....	4.44	58	2.3
Nov.....	1.44	36	2.9	Nov.....	3.08	48	3.0
Dec.....	(0.80)	28	(3.0)	Dec.....	2.22	41	3.4
Year.....	46.70	47	3.2	Year.....	61.05	57	3.2

* Unpublished records furnished through courtesy of U. S. Weather Bureau and U. S. Forest Service.

Wagonwheel Gap: Mean of records for two Class A Weather Bureau Stations, on near-by slopes, one having a northern exposure, and the other a southern exposure. Myton: Class A Station: Santa Fe: Records for 1913-14 taken by floating pan, 3 by 3 ft. by 18 in., in reservoir, 1 mile west of city. Class A Station established in open space on edge of city in 1916. Farmington: Floating pan, 3 by 3 ft. by 18 in., on slough near city. Piute Dam: Class A Station in Sevier River bottom, 8 miles south of Marysville. Piute Reservoir, 500 ft. south, and Sevier River, 200 ft. southeast. Mesa: Class A Station in alfalfa field, 1 mile west of Mesa, in Salt River Valley. Lees Ferry: Class A Station in canyon of Colorado River, 10 miles south of Utah line. Walls of canyon distant 100 and 200 yd., respectively; river, 400 to 600 ft. wide and 140 ft. distant from pan. Willcox: Class A Station in alfalfa field, 3 miles northwest of town, in north central part of Sulphur Springs Valley, which has nearly level floor, 9 miles wide. Deming: Floating pan, 3 by 3 ft. by 18 in., in pond of considerable size. Yuma Reservoir: Floating pan, 3 by 3 ft. by 18 in., on railroad reservoir in Yuma. Values in parentheses were estimated.

TABLE 7.—(Continued).

Month.	Reservoir equivalent, in inches.	Temperature, in degrees, Fahrenheit.	Wind velocity, in miles per hour.	Month.	Reservoir equivalent, in inches.	Temperature, in degrees, Fahrenheit.	Wind velocity, in miles per hour.
MESA, ARIZ., 1917-1925.				DEMING, N. MEX., 1915-1925.			
Jan.....	1.85	49	1.6	Jan.....	2.60	43	...
Feb.....	2.44	54	1.9	Feb.....	3.12	46	...
Mar.....	3.82	57	2.0	Mar.....	5.48	53	...
Apr.....	5.31	63	2.2	Apr.....	6.78	60	...
May.....	6.99	72	1.9	May.....	7.39	66	...
June.....	7.53	82	1.7	June.....	7.97	70	...
July.....	6.91	86	1.5	July.....	6.08	73	...
Aug.....	5.51	84	1.1	Aug.....	5.42	72	...
Sept.....	4.82	78	0.9	Sept.....	5.42	69	...
Oct.....	3.20	68	1.0	Oct.....	4.84	63	...
Nov.....	2.18	57	1.4	Nov.....	3.49	51	...
Dec.....	1.62	50	1.5	Dec.....	2.66	45	...
Year.....	51.68	66	1.6	Year.....	61.20	59	...
LEES FERRY, ARIZ., 1922-1925.				YUMA RESERVOIR, ARIZONA, 1903.			
Jan.....	1.15	33	1.4	Jan.....	3.02
Feb.....	2.07	43	1.4	Feb.....	3.17
Mar.....	3.84	50	2.9	Mar.....	4.80
Apr.....	5.21	59	2.8	Apr.....	6.56
May.....	7.79	72	2.7	May.....	8.95
June.....	8.72	80	2.4	June.....	9.33
July.....	8.87	86	2.1	July.....	9.78
Aug.....	7.04	81	1.9	Aug.....	9.73
Sept.....	5.76	74	1.7	Sept.....	8.24
Oct.....	4.65	60	1.9	Oct.....	5.27
Nov.....	1.96	47	1.5	Nov.....	3.46
Dec.....	1.32	37	1.8	Dec.....	3.08
Year.....	58.38	60	2.0	Year.....	75.39

It will be noted from Table 5 that the factors for the Class A pan, floating, and set in the ground, show a relation comparable with that cited by the author at Nelson Reservoir, Montana.

The writer has recently made a study of the evaporation records in, and adjacent to, the Colorado River Basin and has reduced to reservoir equivalents a number of the records presented by the author (Table 1*), who used data to 1923, inclusive, in determining his mean monthly values. In the following table of reservoir equivalents, Table 6, the means are based on all records to 1925, inclusive. The addition of two years more makes very little change in the mean monthly temperatures and wind velocities as originally given, and they are not repeated.

In addition to the Reclamation records, the writer has compiled records (Table 7), from nine evaporation stations in the Southwest, maintained by various organizations, giving the mean monthly reservoir equivalents of these records, together with temperature and wind velocities.

On an average 50% of the evaporation at the stations noted in Table 7 occurs during the 4-month period from June to September.

* *Proceedings, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 44.*

The evaporation at the higher altitudes is influenced greatly by the slope on which the records are taken. The Wagonwheel Gap records (Elevation 9 610) are the combined results of measurements made on a slope having a northern exposure and one having a southern exposure. The former receives the direct rays of the sun for a shorter period than the latter, and the resulting difference in temperature and relative humidity is strikingly shown by the evaporation on the two slopes during the period from June to October, for which it is possible to measure evaporation without interference from freezing. Table 8 shows the mean monthly values, each covering records of 5 years, for the two slopes.

TABLE 8.—EVAPORATION (RESERVOIR EQUIVALENT) ON SLOPES OF NORTHERN AND SOUTHERN EXPOSURE AT WAGONWHEEL GAP, COLO.

Month.	RESERVOIR EQUIVALENT, IN INCHES.	
	Northern exposure.	Southern exposure.
June.....	2.91	3.82
July.....	2.39	3.69
August.....	1.59	3.12
September.....	1.23	2.96
October.....	0.45	1.89
Total.....	8.57	15.48

Reservoirs of any considerable size at the higher altitudes are usually surrounded by slopes of both exposures, and the evaporation from the water surfaces will approximate the mean of the values for each slope.

In the arid Southwest the factors influencing evaporation for any given month have a relatively small variation from year to year, and this causes a small variation in the total evaporation for each year. Table 9 shows the percentage of mean evaporation measured each year at Farmington, N. Mex., and Mesa, Ariz.

TABLE 9.—VARIATION IN ANNUAL EVAPORATION AT FARMINGTON, N. MEX., AND MESA, ARIZ.

Year.	Evaporation, in inches.	Percentage of mean.	Year.	Evaporation, in inches.	Percentage of mean.
FARMINGTON, N. MEX.			MESA, ARIZ.		
1915	39.11	85
1916	43.34	94
1917	45.69	99	1917	44.21	86
1918	44.18	96	1918	51.52	100
1919	48.57	106	1919	51.36	101
1920	45.01	98	1920	54.76	106
1921	45.58	99	1921	58.63	114
1922	47.21	102	1922	51.37	99
1923	47.64	102	1923	52.85	102
1924	51.71	112	1924	54.97	116
1925	52.62	114	1925	46.13	89

The author calls attention to the need for observations of relative humidity data at evaporation stations. In none of the records given by him are strictly comparable relative humidity records available, as the only records presented were taken some distance away under different conditions. Being at regular U. S. Weather Bureau stations, they were presumably taken at a point some distance above the ground where the air is dryer than close to the evaporating water surfaces.

The writer agrees thoroughly with the statement that a careful study of meteorological conditions must be made before evaporation data obtained in one locality can be used elsewhere. Unfortunately, records of wind velocity obtained at regular U. S. Weather Bureau stations are not directly comparable with wind velocities recorded at the Class A Weather Bureau evaporation stations. The former are usually taken on the roofs of buildings from 35 to 60 ft. above the ground and the latter are taken 2 or 3 ft. above the ground. Table 10 shows a few comparisons between wind velocities in the two classes of records.

TABLE 10.—RELATION OF WIND VELOCITIES AT DIFFERENT ELEVATIONS.

Station.	WIND VELOCITY, IN MILES PER HOUR.	
	Near ground.	Regular Weather Bureau.
Santa Fe, N. Mex.....	2.7	7.1
Yuma, Ariz., evaporation.....	1.4	5.4
Mesa, Ariz.....	1.6	5.2
Roosevelt, Ariz.....	1.7	5.2
Agricultural College.....	2.4	7.5

The chief factors influencing the rate of evaporation are relative humidity, temperature, and wind velocity, and these factors are so interdependent that it is impossible to make, with any considerable degree of accuracy, a comparison between evaporation in different localities, based on a single factor.

Since the rate of evaporation depends primarily on the capacity of the air to absorb additional moisture which, in turn, is measured by the relative humidity, it is possible that if records of the humidity, taken close to the water surface, were available, a relationship between it and the rate of evaporation might be found. Comparable records of relative humidity, however, are practically non-existent.

The interdependence of temperature, relative humidity, and wind velocities is well illustrated by the three Yuma records. While the mean annual temperature at the different stations varied by only 4°, the reservoir equivalent of the observed evaporation varied from 53.8 to 81.3 in. Although records of relative humidity are not available, it is known that they must have been considerably higher at the evaporation station in an alfalfa field near the river than at the railroad reservoir, and still higher than at the citrus station situated on a barren mesa having a wind velocity 85% greater.

R. I. MEEKER,* M. Am. Soc. C. E. (by letter).†—The compilation on evaporation given by Mr. Houk fills a large void in engineering literature, and his extensive tabular data should find wide use, when reduced by the proper coefficients to open-water conditions. One valuable feature of the evaporation records on Reclamation projects is their general standardization; another important feature is the duration of the records, largely ranging from five to sixteen years. Hereafter there can be little excuse in the Western United States for the evaporation vagaries resorted to in the Lake Conchos studies of 1915,‡ and the Southeastern Oregon Duty of Water Analysis in 1918.§

It is to be regretted that a few additional pages were not devoted to the relation of evaporation data to open-water surfaces, together with the necessary coefficients for reducing various types of pans and tanks, to a common basis. While such material is meager, it is correspondingly valuable. There is a pressing need for additional field laboratory studies to check and confirm the present incomplete data on coefficients of reduction, for pans and tanks of various types, to open-water surface conditions.

Epecially is a check needed on the Sleight data secured at the Denver Field Laboratory in 1916.|| Such check work should be done in a different climatic area. It is especially desirable that the relation of data secured by Class A land pans be checked, and that the relation of Class A floating pans be determined for open-water conditions.

Coefficients of Reduction for Evaporation Records to Open Water Surfaces.—During 1916, R. B. Sleight, Assoc. M. Am. Soc. C. E., Irrigation Engineer, U. S. Department of Agriculture, conducted an elaborate series of field laboratory studies of evaporation from water surfaces in evaporation pans and tanks of various shapes and sizes, and under diverse conditions. These field studies were made at Denver, Colo., and are, as far as the writer knows, the most comprehensive data extant in this field of endeavor. The field laboratory was situated in open prairie country, and the investigations covered a period of one year.¶

Table 11 contains coefficients of reduction for evaporation records to open-water surfaces, taken therefrom. In three instances coefficients of reduction have been interpolated by the writer from the Sleight data and are properly noted. Mr. Sleight qualifies** the use of his coefficients of reduction as follows:

"Data on such tanks may be safely extended to large open water surfaces under exactly the same conditions of wind, air temperatures, and relative humidity, by multiplying the evaporation depth by the various coefficients."

* Special Deputy State Engr., Denver, Colo.

† Received by the Secretary, February 6, 1926.

‡ "A Study of the Depth of Annual Evaporation from Lake Conchos, Mexico," by Edwin Duryea, Jr., M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E., *Transactions, Am. Soc. C. E.*, Vol. LXXX (1916), p. 1829.

§ "Determination of the Duty of Water by Analytical Experiment," by W. C. Hammatt, M. Am. Soc. C. E., *Transactions, Vol. LXXXIII* (1919-20), p. 200.

|| "Evaporation from the Surfaces of Water and River Bed Materials," by R. B. Sleight, Assoc. M. Am. Soc. C. E., *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

¶ *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

** Loo, *cit.*, p. 237.

TABLE 11.—COEFFICIENTS OF REDUCTION, EVAPORATION RECORDS TO OPEN-WATER SURFACES.*

LAND TANKS AND PANS.				Remarks.
Depth of tank or pan, in feet.	Depth of water in tank or pan, in feet.	Dimensions of tank or pan, in feet.	Coefficient of reduction to open water surfaces.	
		Circular:		
3.0	2.75	2.0, diameter	0.77	Tank set in ground to depth 2.75 ft.
3.0	2.75	4.0, diameter	0.84	Tank set in ground to depth 2.75 ft.
3.0	2.75	6.0, diameter	0.90	Tank set in ground to depth 2.75 ft.
3.0	2.75	9.0, diameter	0.98	Tank set in ground to depth 2.75 ft.
3.0	2.75	12.0, diameter	0.99	Tank set in ground to depth 2.75 ft.
2.5	2.25	6.0, diameter	0.89†	Tank set in ground to depth 2.17 ft.
2.0	1.75	6.0, diameter	0.88	Tank set in ground to depth 1.75 ft.
0.83†	0.62	4.0, diameter	0.66	Pan set on timbers on ground. This pan is used at Class A evaporation stations, U. S. Weather Bureau.
3.0	2.75	Cubical: 3 by 3, square	0.80	Tank set in ground to depth of 2.75 ft. Fort Collins type.
FLOATING PANS.				
		Square:		
1.5	1.25	3 by 3, square	0.91	Pan set in water and protected by raft to reduce wave action. This is U. S. Geological Survey floating standard.
		Circular:		
0.83†	0.58	4.0, diameter	0.92±‡	Pan set in water; submerged to a depth of 7 in.
1.88	1.50±	3.5, diameter	0.91†	

* Data from Denver Field Laboratory located at Elevation 5346, open prairie land. Long-time climatic records at U. S. Weather Bureau in down-town section of Denver, located on roofs of office buildings, are as follows:

Mean annual temperature, in degrees Fahrenheit.....50.1....53 years record

Wind, in miles per hour.....7.4....."

Relative humidity, percentage.....53....."

Mean annual precipitation, in inches.....14.27....."

† 10-in. depth.

‡ Approximate interpolation by writer from Sleight data.

Confirmatory Data on Coefficients of Reduction for Evaporation Records Secured in Tanks and Pans.—Confirmatory data on coefficients of reduction for various types of evaporation pans and tanks are meager. Table 1* contains records of an evaporation station on Nelson Reservoir, Milk River Project, Montana, which offers a rough confirmation of the Sleight coefficient of 0.66 for 4-ft. circular land pans, 10 in. deep.

At the Nelson Reservoir Station simultaneous records were secured in both land and floating pans of the same size (4-ft. circular, 10 in. deep) for a period of 6 months, covering 3 years. The Nelson Reservoir data give a coefficient of 0.68 for the reduction of records secured in a land pan 4 ft. in diameter to floating pans of the same dimensions. This figure is only roughly comparable to Sleight's coefficient of 0.66 because the floating-pan record is not the equivalent of an open-water surface, but is still subject to a slight reduction to represent open-water evaporation.

* *Proceedings, Am. Soc. C. E.*, January, 1926, Papers and Discussions, p. 50.

Evaporation Data Reduced for Application to Open-Water Surfaces.—Table 12 contains evaporation data reduced for application to open-water surfaces. It is derived from the evaporation records submitted by Mr. Houk and from the evaporation records submitted by the writer in Table 13. Columns (4) and (7) contain the important data used in reduction to open-water surfaces. Column (4) gives average yearly records of evaporation, transposed from depths, in inches, to depths, in feet. Column (7) gives the coefficient of reduction, and Column (8) gives the evaporation values for open-water surfaces, and is the result of applying the coefficients in Column (7) to Column (4).

At the risk of incurring criticism because of the paucity of reliable information concerning proper evaporation pan coefficients, the writer has prepared Table 12 primarily as a step in the correlation of evaporation records with mean annual temperatures. Records from Mr. Houk's paper have been used only where there are complete yearly records for a period of several years.

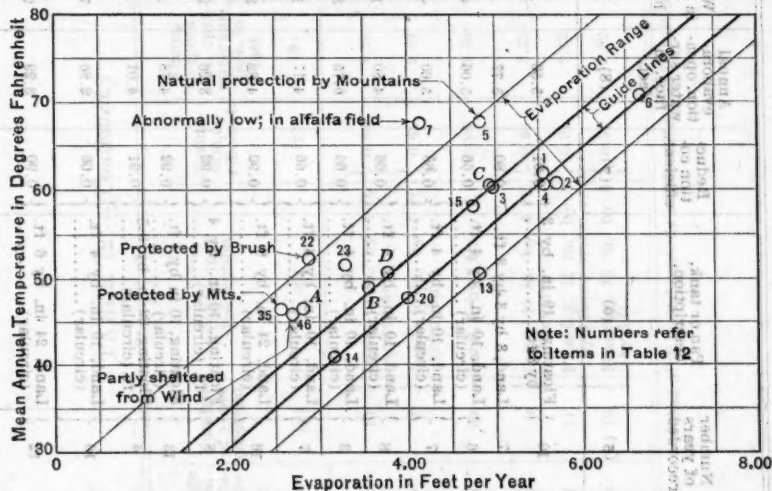


FIG. 5.—CORRELATION OF EVAPORATION WITH MEAN ANNUAL TEMPERATURES FOR OPEN WATER SURFACE IN WESTERN UNITED STATES.

Table 12 contains the information used in Fig. 5. This diagram is a correlation of evaporation with mean annual temperatures and contains "guide lines" for the determination of yearly evaporation values from open-water surfaces. It should be clearly understood that the "guide lines" represent average conditions, and that the plotting of individual years with mean annual temperatures would show a wider dispersion. The heavy straight lines represent approximately average conditions of evaporation for various mean annual temperatures. To the right and left of these heavy lines are lighter parallel lines which roughly set the limit of maximum and minimum values. A study of Table 12 will show that the minimum line is controlled by data at stations where evaporation values are largely reduced because of special protection either artificial or natural. The maximum and minimum lines define graphically an evaporation belt.

TABLE 12.—EVAPORATION SUMMARY WITH CONVERSION TO OPEN-WATER SURFACE.*

Number.	Station and location.	Mean annual temperature, in degrees Fahrenheit.	Average annual evaporation, in feet.	Number of years recorded.	Pan or tank, description.	Reduction coefficient.	Annual evaporation, open-water surface, in feet.	Wind, in miles per hour.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Avalon Reservoir, Carlsbad Project, New Mexico.	62.1	6.11	10	Floating, 19 in. by 3 by 3 ft.	0.91	5.56	Pan anchored near concrete wall.
2	Agricultural College, Las Cruces, New Mexico.	60.3	7.21	7	Land, 3 by 3 by 3 ft.	0.80	5.77	Pan set 3 ft. in ground. Water kept within 6 in. of top.
3	McAllister Park Agricultural College, New Mexico.	60.0	7.67	6	Land, 10 in. by 4 ft. (circular)	0.66	5.06	4.15	Pan on cultivated land on campus. Small trees near-by.
4	Elephant Butte, Rio Grande Project, New Mexico.	61.1	8.48	7	Land, 10 in. by 4 ft. (circular)	0.66	5.60	4.52	Pan on desert hill near reservoir.
5	Roosevelt, Salt River Project, Arizona.	67.3	7.27	8	Land, 10 in. by 4 ft. (circular)	0.66	4.80	1.73	Bugged mountains enclose lake on south and west. Pan on steep slope about 300 ft. from lake shore; 1 mile east of dam.
6	Yuma Citrus, Yuma Project, Arizona.	70.7	10.23	3	Land, 10 in. by 4 ft. (circular)	0.66	6.75	3.03	Pan 8 miles southwest of Yuma, on Desert Mesa.
7	Yuma Evaporation, Yuma Project, Arizona.	67.4	6.32	7	Land, 10 in. by 4 ft. (circular)	0.66	4.17	1.38	Pan in alfalfa field about 1 mile west of Yuma on nearly level mesa. Relative humidity, 44.9%, taken at Yuma.
13	Fallon Experiment Farm, Newlands Project, Nevada.	50.6	5.30	16	Land, 24 in. by 6 ft. (circular)	0.90	4.82	3.24	Pan set 21 in. in ground. Wind records commenced in 1912.
14	Lake Tahoe, Newlands Project, California.	41.8	3.54	8	Floating, 10 in. by 4 ft. (circular)	0.92	3.26	2.65	Pan submerged 7 in. in river, 125 ft. from shore and 60 ft. from dam. Partly sheltered from wind.
15	East Park, Orland Project, California.	58.2	5.20	13	Floating, 10 in. by 4 ft. (circular)	0.92	4.78	Anchored near reservoir shore. No wind obstructions.
20	Klamath Falls, Klamath Project, Oregon.	47.7	4.41	4	Floating, 22 in. by 8.5 ft. (circular)	0.91	4.01	In river about 500 ft. below Main Canal.
22	Cold Springs, Umatilla Project, Oregon.	51.9	4.34	10	Land, 10 in. by 4 ft. (circular)	0.66	2.86	7.07	Temperature at Hermiston. Anemometer 17 ft. above ground. Pan near edge of reservoir on ground. Some brush and willows near-by.
23	Hermiston Experiment Farm, Umatilla Project, Oregon.	51.6	3.66	12	Land, 24 in. by 6 ft. (circular)	0.90	3.29	2.95	Anemometer 3 ft. above ground.

* Column (8) gives evaporation values for open-water surfaces. The values in Columns (3) and (8) are used on Fig. 5.

TABLE 12.—(Continued.)

Number.	Station and location.	Mean annual temperature, in degrees Fahrenheit.	Average annual evaporation, in feet.	Number of years recorded.	Pan or tank description.	Reduction coefficient.	Annual evaporation on water surface, in feet.	Wind, in miles per hour.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
35	Shoshone Dam, Shoshone Project, Wyoming.....	46.5	3.89	10	Land, 10 in. by 4 ft. (circular).....	0.66	2.57	Pan on ground, 1916 to 1918; 16 in. in ground, 1915; on roof, 1919 to 1922; and on ground, 1923 and 1924.
46	Sunflower Camp, North Platte Project, Nebraska.....	46.2	4.07	8	Land, 10 in. by 4 ft. (circular).....	0.66	2.69	7.90	Pan on wood foundation just above ground, partly sheltered from wind.
A	Fort Collins Agricultural College, Colorado.....	46.5	3.50	39	Land, 8 by 3 ft.....	0.80	2.80	6.10	From Table 13. Relative humidity, 67.5 per cent. Tank set in ground with 4-in top rim exposed. Water carried at ground surface. Abnometer 60 ft. above ground.
B	Santa Fe Field, U. S. Weather Bureau, New Mexico.....	48.3	5.33	9	Land, 10 in. by 4 ft. (circular).....	0.66	3.55	2.68	Pan in open field in northwestern part of Santa Fe. Relative humidity, 54.5 per cent.
C	Deming, State Engineer, New Mexico.....	60.6	5.52	11	Floating, 11 in. by 3 by 3 ft.....	0.91	5.02	At Cobbs Ranch; temperature of water in pan, 60.2°; outside of pan, 53.3 degrees.
D	Farmington, State Engineer, New Mexico.....	51. *	4.20	10	Floating, 18 in. by 3 by 3 ft.....	0.91	3.32	Pan in slough on Wahls Ranch. Temperature of water in pan, 55.1°; outside of pan, 55.1°.

* Mean of 20 years' records at Bloomfield and Fruitland.

* Column (8) gives evaporation values for open-water surfaces. The values in Columns (3) and (8) are used on Fig. 5.

TABLE 13.—EVAPORATION RECORDS, PLAINS AND PLATEAU AREAS, WESTERN UNITED STATES.

Station State..... Exposure..... Pan or tank..... Elevation..... Records..... Maintained by..... Remarks.....	A.				B.				C.			
	Evapora- tion, in inches.	Tempera- ture, in degrees Fahren- heit.	Wind, in miles per hour.	Relative humidity, percen- tage.	Evapora- tion, in inches.	Tempera- ture, in degrees Fahren- heit.	Wind, in miles per hour.	Relative humidity, percen- tage.	Evapora- tion, in inches.	Tempera- ture of water in pan, in degrees Fahren- heit.	Tempera- ture of water out- side of pan, in degrees Fahren- heit.	Tempera- ture, in degrees Fahren- heit.
Fort Collins Colorado Land 3 ft., cubical 5 000 1887 to 1925, inclusive Colorado Agricultural College Tank set in ground with 4 in. of top rim ex- posed; water kept at ground surface. Tank located on campus; 4 different locations in 39 years. Anemometer located on ground at evaporation tank. Records, 1911-25.	1.41 1.51 2.63 4.29 5.61 5.78 5.33 3.25 1.58 1.29	35.9 27.4 35.7 45.6 54.0 63.4 67.3 59.2 47.5 35.8 27.7	2.1 2.0 2.6 2.8 2.0 1.6 0.85 0.96 1.3 1.6 1.8	71.5 72.4 66.7 60.2 63.1 63.6 68.5 67.8 68.4 70.4 72.8	1.63 2.00 3.32 6.07 8.78 10.38 8.83 6.60 4.60 2.56 1.83	23.8 23.7 35.2 45.5 55.5 65.6 69.1 67.8 61.3 50.1 39.1 31.1	2.9 3.1 3.6 3.9 3.5 2.9 1.9 1.6 2.1 2.4 2.4	64.2 62.3 54.6 47.8 43.9 33.4 33.4 36.6 53.9 57.0 65.2	2.63 2.41 5.94 7.14 8.19 8.78 7.14 5.88 5.55 3.54 2.63	44.4 45.5 53.0 55.2 65.2 71.7 73.5 73.5 70.7 53.2 44.5	43.9 47.5 50.9 58.0 66.5 76.6 77.2 71.0 62.3 51.8 43.4	60.6
Totals and means...	41.96	46.5	1.7	67.5	64.55	48.8	2.68	54.5	66.29	60.2	59.3	60.6

Deming
New Mexico
Floating
16 in. by 3 ft., square
4 300
1914 to 1924, inclusive
State Engineer, New Mexico
Pan located on small body of water at Cobb's
Ranch. Air temperatures taken at Deming
by U. S. Weather Bureau.

Santa Fe Field
New Mexico
Land
8 975
1917 to 1925, inclusive
U. S. Weather Bureau
Class A station located in open field (unirri-
gated) in northwestern part of Santa Fe.
Relative humidity taken at Weather Bureau
Station ¼ mile southeast. Anemometer just
above evaporation pan 16 in. above ground.

TABLE 13.—(Continued.)

Station	D.	E.	F.				
			Evapora- tion, in inches.	Tempera- ture of water in pan, in degrees Fahren- heit.	Tempera- ture of water out- side of pan, in degrees Fahren- heit.	Relative humidity, percent- age.	Wind, in miles per hour.
Farmington State.....	New Mexico	North Platte Nebraska Land 3 000 30 in. by 6 ft., circular	0.86	33.6	38.4	7.1	7.1
Exposure.....	Floating	1907 to 1925, inclusive	1.41	41.0	47.1	7.9	7.9
Pan or tank.....	5 800 ±	University of Nebraska	3.25	41.0	47.1	7.9	7.9
Elevation.....	5 800 ±	1907 to 1925, inclusive	5.80	64.7	68.3	11.0	11.0
Records.....	1915 to 1924, inclusive	Tank located on tableland near Experiment Station. Set in ground to average height of water surface. Water kept to within 2 to 4 in. of top of tank. Wind velocities at sub- station.	7.67	64.7	68.3	11.0	11.0
Maintained by.....	State Engineer of New Mexico		7.67	71.0	73.8	9.2	9.2
Remarks.....	Pan located in slough at Wahls Ranch		7.67	75.5	78.7	6.8	6.8
			6.17	72.4	75.7	9.7	9.7
			4.74	63.2	66.4	7.3	7.3
			3.67	54.5	57.1	7.9	7.9
			1.94	43.8	46.0	7.9	7.9
			0.99	37.6	39.4	8.6	8.6
Totals and means...	50.36		52.66	63.9	63.9	7.0	7.0

* Annual temperature is average of 20-year periods at Bloomfield, 12 miles east, and Fruitland, 12 miles west, of Farmington.

The upper end of the heavy "guide lines" is controlled by data for mean annual temperatures between 60 and 70°, and are applicable to Imperial Valley conditions. The lower end of the heavy "guide lines" is controlled by evaporation values for mean annual temperatures between 40 and 50 degrees. The evaporation values taken from Fig. 5 are gross values, and in applying them to reservoir studies, yearly quantities, in feet, should be reduced by subtraction of yearly precipitation values in order to determine net evaporation values for the water surface under consideration.

The straight line relation, defined by the "guide line", is not offered as a panacea for any and all evaporation determinations where no records are available. Fig. 5 is offered as a guide for the determination of maximum and minimum limits as correlated with mean annual temperatures, and furnishes the ground for a decision as to the probable average yearly values, assuming, of course, that consideration be given to local conditions obtaining at the reservoirs or bodies of water where evaporation values are desired. Where time and other considerations preclude the actual determination by field laboratory methods of evaporation values, it may aid to define limits and set an approximate figure based on mean annual temperatures.

A study of evaporation data in the paper, and elsewhere in engineering literature, shows the danger of making blind applications of existing records to other areas without due consideration of local climatic and physical conditions in both the areas of origin and application. Intelligent interpretation of evaporation data requires careful study of the conditions under which evaporation observations have been made.

This discussion is intended as a preliminary effort at correlation of evaporation records in the Western United States. It is hoped that Fig. 5 may serve as an engineering tool and guide to further efforts and studies along this line of engineering research.

Table 13 is submitted as additional information to supplement Table 1. Messrs. Houk and Debler have performed a notable task in compiling evaporation records for forty-six stations; they deserve the thanks of the Engineering Profession.

B. E. TORPEN,* M. AM. SOC. C. E. (by letter).†—This subject is of general interest in the West. All available data should be forwarded to the Special Committee on Irrigation Hydraulics for its use. From all the data collected it is hoped the Committee will be able to formulate definite rules and instructions for the use of, say, three standard types of evaporation pan installations, namely, floating, on the ground, and 10 ft. above the ground, and specify a factor to use with each type in determining the probable evaporation from a storage reservoir.

From Table 2,‡ it is apparent that a single year's record will probably be within a range of 12.5% of the yearly mean. Such a record, therefore, has great value in estimating the evaporation from a proposed reservoir.

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† Received by the Secretary, February 8, 1926.

‡ *Proceedings*, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 56.

The Cushman Storage Reservoir, with a capacity of 450 000 acre-ft., has a maximum surface area of 4 000 acres and a minimum area at full draw-down of 2 000 acres, with a probable yearly mean area of 3 650 acres. In hydraulic studies of the probable yield of the reservoir for power purposes a mean annual evaporation of 36 in. was assumed to be liberal for the site. This is equal to a continuous flow of 15 cu. ft. per sec. and is almost 2% of the annual mean run-off of 800 cu. ft. per sec.

TABLE 14.—EVAPORATION FROM CUSHMAN STORAGE RESERVOIR.

Station: Lake Cushman. State: Washington. Project: Cushman Power Project. Exposure: Floating. Dimensions of Pan: 10 Inches by 4 Feet, Circular. Elevation: 760. Records: 1925 (Water Year) October, 1924 to September, 1925. Maintained by: City of Tacoma. Remarks: No Wind Obstructions; Pan Kept Half Full; 6-Inch Stilling Well in Center of Pan.					
MEAN RECORDS.					
Month.	Evaporation, in inches, at Cushman.	Temperature, in degrees Fahrenheit, at Cushman.		Wind, in miles per hour, at Tacoma.	Relative humidity, percentage, at Tacoma.
		Water.	Air.		
January.....	0.05	39	40	10.6	85.0
February.....	0.25	39	43	8.9	74.5
March.....	1.47	45	46	6.9	67.0
April.....	1.76	50	47	5.9	70.5
May.....	4.02	62	62	7.2	58.5
June.....	4.75	67	65	7.4	55.5
July.....	5.50	68	67	6.0	58.5
August.....	3.92	65	58	7.4	59.0
September.....	3.01	58	48	5.6	62.5
October.....	1.17	48	46	8.8	70.5
November.....	0.31	40	42	7.5	81.0
December.....	0.06	38	35	8.7	89.0
Totals and means.	26.27	51.1	49.5	7.58	68.5
					101.03

The year's record (Table 14) taken at Cushman for a check on this assumption gives a floating-pan evaporation of 26 in. and indicates that the 36 in. assumed was high. It is probable that the evaporation from the reservoir is not the same as from a floating pan. The writer hopes the Committee will be able to arrive at the proper factor to apply to pans in determining the actual evaporation from a reservoir.

**HISTORY AND PROBLEMS
OF IRRIGATION DEVELOPMENT IN THE WEST**

Discussion*

By MESSRS. SAMUEL FORTIER, ARTHUR P. DAVIS, C. E. GRUNSKY,
AND F. H. NEWELL.

SAMUEL FORTIER,† M. A. M. Soc. C. E. (by letter).‡—What may be regarded as the most general irrigation policy of the United States is that the cost of building, maintaining, and operating irrigation works is a direct charge against the land benefited. The general character of this policy may be better understood by a brief reference to the methods followed by each of the several agencies engaged in irrigation development. These include (1) individuals and groups of individuals; (2) co-operative or mutual companies; (3) commercial companies; (4) irrigation districts; (5) the U. S. Reclamation Service now the Bureau of Reclamation; (6) Carey Act companies; and (7) the U. S. Indian Service. It is obvious that the first two named, being composed of farmers acting singly, in partnerships, or in organized bodies, pay for their irrigation systems. The commercial company constructs and operates the canal system but distributes both costs over the land under it; formerly it was the custom to secure re-payment for the works by the sale of water rights and to maintain and operate the system by the annual collection of water rentals. The sale of water rights having been prohibited by the laws of many Western States, commercial companies now deliver water to farmers at rates fixed usually by some public authority. Under authority conferred by the State the irrigation district may bond the lands of the district and in this way secure money to buy or build works. Sooner or later, however, those who own the irrigable land within the district are obliged to pay all the indebtedness incurred in building and maintaining irrigation works.

The same principle is embodied in both the Carey and Reclamation Acts. In the former, the Federal Government donates the land to the State on condition that it be reclaimed. The State in turn makes use of the corporation as a reclamation agent to provide capital, construct works, and pro-rate the cost over the lands reclaimed. Under the terms of the Reclamation Act the Government takes the place of the corporation in the Carey Act in financing

* This discussion (of the paper by John A. Wiltsoe, Esq., presented at the Summer Meeting, Salt Lake City, Utah, July 8, 1925, and published in March, 1926, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, July 8, 1925.

and constructing works, but the expenditures, less the interest charge, are assessed against the lands benefited and each entryman assumes his *pro rata* share. About the only exception to this rule is the Indian and even he, when possessed of the necessary funds, comes under this country-wide policy since a large part of the funds expended by the U. S. Indian Service on the construction of irrigable works is reimbursable to the Government.

This policy is not only general but fundamental, for so long as it is in effect irrigation works and irrigation systems, regardless of their cost or magnitude, should properly be classified with other property belonging to farmers. This conception of irrigation—and it is believed to be the true one—brings the construction of irrigation works into close relationship to agriculture and renders necessary as a preliminary measure a critical study of such questions as the property of the farmer; its character as regards cost, durability, and efficiency; the profits to be derived from farming; and the ability of the average farmer to redeem a farm mortgage.

According to the U. S. Census of 1920 the total values of different kinds of farm property were, in round numbers, as follows:

Land	\$54 830 000 000
Buildings	11 486 000 000
Implements and machinery.....	3 595 000 000
Domestic animals	8 013 000 000

There are nearly 6 500 000 farms in the United States and the mind cannot well grasp their aggregate value; but some idea of their true status is readily obtained in dealing with averages. Thus, the average value per farm in 1920 was \$8 503 for land; \$1 781 for buildings; \$557 for implements and machinery; and \$1 243 for live stock. The average mortgage indebtedness as accurately as it could be ascertained was \$3 356, leaving an equity of \$8 728 per farm. A further analysis of farm property shows that the farm mortgage debt nearly quadrupled from 1890 to 1920; that farm dwellings as a general rule are inconvenient and unsanitary, and are lacking in both comfort and taste; that the buildings which house stock and protect implements and machinery are inferior in quality and poorly maintained; and that, in appearance, character, and efficiency, farm structures generally are of a low order when compared with those of civic and industrial development.

In regard to the average net income of the farmer in the United States, the preponderance of evidence goes to show that it is low. According to data collected by the U. S. Department of Agriculture prior to 1917, on 266 farms in New Hampshire the farmer received on an average \$337 for his year's work. In fairly prosperous rural communities in Chester County, Pennsylvania, the average yearly income of 378 farms was \$789. In three districts located in Indiana, Illinois, and Iowa, respectively, the average farm income was \$870, and the average income on 69 irrigated farms in the Salt Lake Valley, Utah, was \$417. To these incomes should be added the food and fuel derived from the farm and consumed by the family living thereon as well as

the shelter furnished by the farm dwelling. According to the same authority the average total farm income in 1922 on 6 094 farms located in different parts of the country was \$1 211, made up of a net cash income of \$715, value of food and fuel produced on the farm, \$294, and an increase in inventory of \$202.

Some idea of the gross returns per acre per annum from irrigated land may be had from the records of Government irrigation projects, chiefly for the ten-year period from 1912 to 1922. These records cover 26 projects in 14 Western States and average in gross value \$45.66 per acre.

In contrasting the achievements in irrigation development during the past 75 years with the policy herein outlined, it is at first difficult to understand how so much could be done with funds derived from the profits of irrigated farms. A brief review of this development and its noteworthy achievements discloses the fact that irrigation farmers have received since the pioneer stage large amounts of capital and a large number of completed or partly completed canal systems at greatly reduced costs from outside sources.

The report of the Census for 1920 shows that more than 19 000 000 acres were irrigated at that time at a cost of about \$700 000 000, but although this cost under the general requirements was to be paid by the profits from irrigation farming, as will be shown later, only a part was so paid.

The pioneer found fertile river bottom-lands that could be watered readily from tributary streams and both the stockman and the farmer were able through their individual efforts, or those of small groups of their class, to establish homes supported in part at least by the products of irrigated farming. The first 1 750 000 acres, approximately, was thus reclaimed without aid from outside sources. Those who owned the land controlled the water and as a rule large profits were derived from a union of cheap land and cheap water. The crude and inexpensive methods of the pioneer, however, were not adapted to the irrigation of the higher bench lands, which frequently involved high diversion dams and long lines of main canals through rocky canyons and along steep mountain slopes. Therefore, when the lower lands bordering the streams had been irrigated another agency had to be found to finance and construct the larger and more costly works for the irrigation of the higher bench lands. It was at this juncture in the late Seventies that the land and water corporation entered the field and for twenty-five years continued to be the main agency in the irrigation development of the West.

It was due to the successful efforts of the pioneers that the productivity of desert lands had been demonstrated and this, in turn led to far-reaching possibilities along the lines of land settlement, rural and urban development, transportation, industry, and commerce—in short, the possibility of building an empire in a land that was thought fit only for the abode of the buffalo and the Indian. This was the lure that attracted capitalists, who thought they saw wonderful opportunities for profitable investment. Promoters organized companies, secured capital from America and foreign countries, filed on water supplies, acquired irrigable lands whenever they could be had at a low price, and proceeded to build canal systems, with the optimistic belief of at-

taining independence in a few years by the sale of water rights and the collection of water rentals. Those who invested money in these enterprises realized after it was too late that the bulk of the lands under these new systems were held by speculators who would neither use nor pay for water, that the *bona fide* settlers were few in number and possessed of little means, that the maintenance and operation of canal systems were costly, and that it was the work of a generation rather than a few years to establish prosperous communities in an irrigated region.

Few of these commercial enterprises returned any interest to the investors and in the large majority of cases much of the principal was lost, but the systems remained and a large percentage has since been acquired by the farmers under them at a valuation much below the original cost.

The financial failures of commercial irrigation enterprises which occurred from 1870 to 1895 were repeated by Carey Act projects from 1898 to 1914. The main provision of the Carey Act is that the Secretary of the Interior is empowered on request of the State to donate to the State not exceeding 1 000 000 acres of land, on condition that it be reclaimed and settled. The States which accepted this offer were either too poor to construct irrigation works, or they were barred from doing so by their Constitutions and they "sidestepped" the issue by contracting with companies to do the work. These companies agreed to build works supplying water for definite areas of land and to sell to settlers water rights carrying an interest in the works. Funds for construction were usually obtained from the sale of stocks and bonds, but all the security the company could offer to purchasers was a lien on the works to be built and on the notes of settlers covering deferred payments on water rights. This later security failed to materialize on many projects for lack of settlers and the result was heavy losses to the investors. It is claimed that only 4% of Carey Act projects were financially successful. Canal systems were built to water about 1 250 000 acres, but in time many of these systems were re-organized into irrigation districts and other enterprises and the land owners within them fell heir to the systems at prices far below the original cost.

By the terms of the Reclamation Act the Secretary of the Interior was authorized to construct works to provide an adequate water supply for the arable lands in each project and to return to the reclamation fund, in ten annual installments or less, by assessments levied on the land owners of each project, the costs incurred in building and maintaining the necessary works.

By the terms of an amendment to the Act passed August 13, 1914, the time of payment was extended to twenty years. These more liberal terms, however, do not account for the lag in refunding construction charges. After twenty-two years of operation of this Act one finds a construction debt of \$148 827 844, out of a total construction cost of \$152 566 386, still due the Government. Under this Act the liberality of the Federal Government in relieving the settler of the payment of all interest on the cost of his water right and its leniency in extending the time of repayment of charges have been of material aid to farmers on Government projects.

In turning from the past, with its long record of successes and failures in irrigation development, to a consideration of the future it would be folly to disregard the knowledge and experience that have been acquired at so great a cost in money, labor, and human suffering. One of the lessons taught by the past is that farmers unaided cannot establish homes, prepare their farms for successful irrigation and profitable crops, and, at the same time, pay for costly water rights. They succeeded during the pioneer stage when land was cheap and the ditches inexpensive, but in the later stages failures have been common notwithstanding the fact, as has been pointed out, that they have received from time to time, from various outside sources, a large amount in the aggregate of financial assistance. If the profits from irrigation farming have not sufficed to pay for water rights in the past when conditions were comparatively favorable there is no probability that they will do so in the future, under much more unfavorable conditions. It is generally conceded that the easy tasks in so far as irrigation development is concerned have been done, the difficult tasks await this and succeeding generations.

A reasonably accurate estimate of the water requirement of crops and stream flow made by the writer in 1924 showed that about 57 000 000 acres might be irrigated in the 17 Western States. According to the Census of 1919 the systems were capable of irrigating 26 000 000 acres. This leaves 31 000 000 acres yet to be reclaimed if one excludes what has been accomplished since 1919. A large part of the water supply for this acreage, providing it is ever reclaimed, will be derived from a storage of flood waters and this fact, coupled with long canals in difficult locations and the preparation of rough and uneven land, will increase the cost of water per acre far beyond that of recent undertakings.

The people of the West may elect to follow one of two courses: They may decide not to reclaim by irrigation any more arid land; or they may decide to grant the necessary State aid for a further extension of irrigated agriculture. The first course would have few advocates since it is not only opposed to the progressive spirit of the West, but it is believed to be unsound economically. Much of the material prosperity of the West is based on irrigated agriculture and if no more land is to be rendered productive, rural growth, industry, and commerce would be at a standstill. A number of favorable conditions and circumstances seem to demand that more food and clothing be produced in the Pacific and Rocky Mountain States. It is a healthy region and its invigorating climate will attract and hold settlers. About three-fourths of the total hydro-electric energy of the nation is to be found west of the Missouri River. Future generations will also look to the West to supply the bulk of the minerals needed in industries, and beyond all is the broad Pacific and commerce with the Orient.

It would seem necessary, therefore, to devise ways and means of establishing more farm homes under new irrigation enterprises and, at the same time, seek to better the condition of those living under the old systems by a more or less complete overhauling of their faulty methods of conveying and distributing water. In the majority of cases the building of new works will be closely linked with the re-organization, enlargement, and reconstruc-

tion of old enterprises; and if this truth becomes generally recognized in time it will greatly aid in the adoption of proper methods and policies for future development. A large number of Western communities in the aggregate have reclaimed as much land as they can with the use of the summer flow of streams. They have made little progress for years owing to the lack of storage of flood waters. With part or all of the available flood waters stored, most of their leaky ditches abandoned, and a much smaller number of good canals built, the irrigable area might be increased from 25 to 100 per cent. These are the nuclei around which much of the extension of the irrigated area is certain to cluster.

This building on the one hand and remodeling on the other calls for new boundaries of irrigation enterprises. By this is meant the water-sheds of streams. All the water users on the smaller water-sheds should be amalgamated under one organization, and those on the larger, under several, divided by natural and political lines of cleavage. At present, outside of California, there is no type of organization adapted to the inclusion of all the irrigable lands of a water-shed when these lands are made up of non-irrigated, partly irrigated, and irrigated holdings. The irrigation district has been and is a wonderful aid in irrigation development, but its provisions are based on the assumption that the lands included come under the same classification of not being irrigated but susceptible of irrigation. What is needed is a more elastic organization that will include dry as well as irrigated lands, individuals as well as co-operative companies.

In providing water supplies for the 30 000 000 acres or more yet to be reclaimed it is doubtful whether the farmers who will eventually occupy these lands will be able to pay more than 50% of the cost of the water supplies. Placing the average cost at the conservative figure of \$100 per acre would involve an outlay of more than \$3 000 000 000. This amount, large as it may seem, is small compared with what the farmers would be required to invest in developing their lands. The land and its preparation, farm systems of irrigation, buildings, livestock, and equipment, for the area under consideration would cost fully \$9 000 000 000. Judged by the past the income from this farm investment would be small, but it would place the West on a sound and prosperous foundation by increasing its urban population and wealth and by greatly enlarging its industries, transportation, and commerce.

During the past 23 years the Federal Government has expended, including interest, about \$170 000 000 in aid of irrigation in fourteen of the Western States. The financial assistance given by any of the Western States during this period or before has been negligible.

The Federal Government should complete the projects it has begun and fulfill its obligations to settlers; but, from the standpoint of fairness to other sections of the country which are in as great need of National aid, it is difficult to justify a continuation of Federal aid for irrigation without full co-operation with the States concerned.

On the other hand the granting of National or State aid to farmers is attended with considerable risk. When done in the wrong way it tends to break down that spirit of sturdy independence which has been inherited from

Anglo-Saxon and other fine lines of ancestors and to transform self-sustaining Americans into bounty-fed dependents. The part performed by the State or the Nation in the irrigation development of the future should be well defined and wholly apart if possible from that performed by the farmers. If this policy is to be followed it would be best for the State, acting alone or in co-operation with the Federal Government, to build the necessary storage works and to wholesale the water thus provided at reasonable rates to organizations of farmers. This was the policy advocated by the writer and many of his colleagues twenty-five years ago, before the Reclamation Act was passed. If the Government had begun at that time and continued until the present to expend the funds derived from the sale of public lands in conserving the water supplies of the West and contracting their use to private irrigation enterprises on terms favorable to the users, much greater progress would have been made and the West would have had fewer farmers spending their working hours writing Congressmen for payment extensions instead of milking their cows.

ARTHUR P. DAVIS,* PAST-PRESIDENT, AM. SOC. C. E.—The statement by Mr. Fortier†, that of the total capital investment in Reclamation works—about \$152 000 000—more than \$148 000 000 is still unpaid, needs further explanation. From his figures it might be inferred that only \$4 000 000 has been repaid to date. This is far from the truth, for as this money is repaid it is immediately re-invested, as provided by law. Normally, therefore, the difference between the capital invested and the amount unpaid will be merely a reasonable working fund as long as the law operates.

The time and place of the origin of irrigation are unknown. The earliest records of Assyria, Babylon, Egypt, Persia, India, and China, and practically every country of antiquity, bear testimony to ancient and well developed practices of irrigation. It is probably one of the oldest occupations of civilized man. It is a well established fact that civilization originated and developed in an arid region. At the time of the Spanish Conquest in Mexico, extensive irrigation systems existed, antedating the earliest traditions of the people using them. Traces of such works were found in South and Central America and in Arizona, New Mexico, Colorado, and California.

As to reclamation as a modern activity of the Anglo-Saxon race, one of the earliest examples is the Salt Lake Valley, Utah. The early settlers of California, Arizona, and New Mexico extended the previous practice of the Spanish and Indians in those States. Under those primitive conditions, only the simplest and easiest irrigation enterprises could be carried out. No others were justified by economic conditions.

Under the physical circumstances of the Salt Lake Valley, where the streams emerge from the high mountains, discharging most of their water, during the summer, in rivers and creeks with rapid fall, traversing wide fertile plains with considerable slope, irrigation presented advantages which were quickly recognized and successfully developed by the Mormon leaders. Under such circumstances anything that would divert a stream could be

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† See p. 755.

used to place the water in a ditch and convey it to lands in the immediate vicinity without any appreciable expense, except a moderate amount of human labor and enterprise.

The hardy pioneers carried out these methods of using the smaller streams on the lower valleys which they traversed, and, in the aggregate, cultivated a great deal of land and produced large quantities of forage. These enterprises extended to all the States in which favorable conditions for easy development could be found, but these sites were gradually exhausted. The unregulated water supply, declining during July, August, and September, would accomplish only a fraction of the total possibilities of the stream, and, to increase this supply, large storage reservoirs were necessary, generally in regions difficult of access and requiring large investments. Other streams flowing through canyons could be utilized only by high diversion dams, long tunnels, pressure pipes across other canyons, and expensive side-hill canal work, sometimes a long distance from the land to be irrigated; and these projects also required storage reservoirs for their full development.

These enterprises, which required such heavy investments, could not be carried out by the individual efforts of the farmers, and private capital was invited to assist in the development. For a period the representations of promoters inspired investors with enthusiasm and confidence, and considerable areas were reclaimed; but the necessity of heavy investment with its promotion and interest burdens, and the long wait for settlers to develop the desert lands before production could begin, soon demonstrated that the rosy anticipations of promoters were not justified. It may be stated that, with few exceptions, investors in irrigation works as such were financially unsuccessful. Although the development produced might be of benefit to the country, those benefits were largely absorbed by the owners of the land, and, if they were not the same persons as the investors in the works, the latter were generally losers.

It took only a few years to demonstrate that such investments were hazardous and most of them disastrous. Then came the era of development under district enterprise. State laws were passed to encourage this method and much progress was made by combining land ownership with irrigation development. Again, however, the heavy interest charges, the long time and the large investment required for full development, made this method slow and burdensome and entirely unsuited to many of the larger and more expensive projects requiring greater capital and long time in construction and development.

The assistance of the Government, through its ownership of the public lands, was early invoked, as manifested by various laws, especially the Desert Land Act, conferring title on the individuals that reclaimed the land. This Act was, in the aggregate, a great assistance to the smaller enterprises, but did not remove the difficulty of development where the enterprises were large and costly, as the land could not, under the law, be turned over to the large interests necessary for its irrigation.

As the easier irrigation projects were selected and developed by one of the various agencies available, the problem became more and more acute as

the physical difficulties of undertaking new projects progressively increased by the elimination of the smaller and easier developments. Recognizing these facts and the interests of the people at large in the irrigation development of the Western States, a movement was started about thirty years ago to adapt the laws to the handling of the proceeds of the public lands in such a manner as more effectively to encourage their development.

In the Middle States the Homestead Act was effective in settling and developing the country under the conditions there obtaining. The settler was given, as a resident, 160 acres of land on condition that he develop it and build a home. The clearing, fencing, and cultivation necessary were made effective by the climatic condition under which a single successful home could be made through the efforts of one family without the co-operation of its neighbors. This law, however, was found to be unadaptable to a country requiring irrigation, especially where extensive investment of cash and labor was required. So far as irrigation from smaller streams could be carried out by a small group of farmers through their combined labor, this was done, but the attempt became impossible when applied to the difficult enterprises requiring large storage reservoirs, expensive dams, canals, tunnels, flumes, and pressure pipes. The same theory, however, could be adapted to this construction; that is, the values residing in the public lands could be devoted to the development of agriculture in the arid States, as well as in the humid States, if the laws were properly drawn for this purpose.

This was the intent in 1902 when the National Reclamation Act was passed appropriating the receipts from the sales of public lands in the sixteen Western States to the construction of irrigation works in that region. Its avowed purpose was to construct the larger and more difficult projects which were incapable of development through private capital or district agencies. In the selection of projects under this law many were rejected on the ground that they were small enough and easy enough for successful handling by private enterprise. In some cases this proved to be a mistake and projects rejected for this reason were undertaken after a trial had demonstrated that they could not be successfully developed by private enterprise.

Under this law, twenty-five projects have been developed, resulting in a full water supply being made available for nearly 2 000 000 acres of land, furnishing homes for 30 000 families, and a supplemental water supply for about 1 000 000 more, or nearly 3 000 000 in all.

To accomplish these results reservoirs have been built with an aggregate capacity of more than 10 000 000 acre-ft.; 15 000 miles of canals and ditches have been constructed; 103 tunnels, aggregating about 150 000 ft. in length, have been built; structures on the canals, such as drops and turnouts, aggregating about 126 000; more than 9 000 bridges have been constructed, aggregating 217 000 ft.; 11 000 culverts have been built, aggregating about 400 000 ft.; 3 160 000 lin. ft. of pipe have been provided; nearly 4 000 flumes, with a total length of 750 000 ft., have been supplied; and about 1 600 buildings for offices, power plants, pumping stations, barns, and storehouses have been constructed. In connection with this work, there have been built 83 miles of railroad, 1 100 miles of wagon road, and 3 284 miles of telephone lines. The

work performed has involved the excavation of more than 200 000 000 cu. yd. of earth, and nearly 50 000 000 cu. yd. of rock and indurated material. It has required 3 500 000 cu. yd. of concrete and a somewhat larger number of barrels of cement. Incidental to this work, a few power plants have been built as by-products of the storage development, the proceeds of which were credited to the water users on the projects. These plants have developed 64 300 h.p. of energy and have required the construction of about 1 500 miles of line to transmit the electrical energy to the point of use. This power development, scattered over many projects, while small compared with the aggregate development, has been an important factor in the comfort and prosperity of the settlers, being mainly used in the homes and towns developed by the irrigation enterprises.

The total investment in National irrigation has been about \$190 000 000, of which about one-third has been from re-payments by the settlers of the money invested for their benefit. It is believed to be the only National construction, except the Panama Canal, which has made such large returns in proportion to the investment.

The cash value of the annual products of these twenty-five projects aggregates more than \$50 000 000, even in this period of agricultural depression, and, in more prosperous times, has approximated \$90 000 000, as in 1919; it is safe to state, therefore, that the product of three years exceeds the investment by the Government, and that the aggregate has been more than \$700 000 000. The estimated increase in land values, due directly to irrigation works, is \$600 000 000, or more than three times the investment. The law requires the return of that investment, with the exception of the money expended on projects which have not been built or completed, and under the liberal terms offered by the Government and the long time for re-payment, this, in most cases, could be done from the products of the land, except for the propaganda against such return on the part of opposing interests. Whether such propaganda will succeed in its endeavor to wreck the irrigation enterprise remains to be seen, but the values have been created by the combined influence of Government investment and the activities of the industrious farmer, and they have added to the wealth of the West what practically amounts to an additional State in value and in product.

From the small beginnings described, irrigation has become an important factor in nineteen States and is the dominant form of agriculture in eleven Western States. These eleven States contribute more than 20% of the value of all crops in the United States, producing an average value per acre of more than double that of the unirrigated farms of the country.

Probably the most serious weakness of the original National Irrigation Law was the requirement, that the major part of the money be expended in the States in which it was received from the sale of public lands. This injected an entirely foreign element into the consideration of projects for construction and produced a still greater moral influence in favor of regarding the Reclamation Fund as a fund for distribution among the States, rather than as one to be allocated in accordance with the business merits of the various projects. This influence led naturally and inevitably to the selection

of projects in States where the public lands were bringing in large revenues, even when those projects were of inferior merit. In the promotion of projects in such States, the promoters dwelt more on the rights of the State to use the fund than on the physical merits of the project advocated.

This provision of the law was repealed in 1910, but not until after most of the projects had been selected and construction started, and a feeling of proprietorship had grown up in each State concerning ownership of the Reclamation Fund. There have not been wanting those who argue that the moneys received in a given State from the sale of public lands naturally belong to that State to be invested there, irrespective of the merits of the proposed investment, and this spirit continues to prevail about as strongly as ever.

Recently, efforts have been made by public officials to induce or require the States to share in the expenditures and responsibilities of making the irrigation enterprises successful through the selection of competent settlers and of supplying them with the means of improving and properly equipping their farms so as to put them on a paying basis. Such measures have been successfully carried out in some foreign countries, and probably with proper supervision the same might have been true in America. The agitation for relief from payments has made such progress and received so much encouragement in official quarters that it is doubtful whether collection could be enforced, if such measures were adopted. The involvement of the State in the responsibilities for the success of the projects in itself is an excellent idea and would contribute greatly to their success.

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E.—That water is the nation's most valuable natural resource requiring conservation is coming to be generally recognized. In the matter of the volume of supply Nature has been generous, but as to distribution to the various sections of the land and as to dependability as regards the time of availability considerable improvement might have been suggested.

Wherever the human race has established itself in permanent settlements water has been needed; first, of course, for domestic purposes, for drinking by man and his domesticated animals; for washing and bathing; to facilitate transportation; for irrigation; and, where available in flowing streams, to generate power, which, in recent times, is made available at places remote from the source, by electrical transmission. The unequal distribution of rain and snow as to time and place, coupled with the orographic features of the surface of the earth, has resulted in great diversity of stream flow, and, therefore, as soon as the demand for water exceeded the supply of the local spring, stream, or well, the newer problems of control, regulation, and diversion of flow—in other words, of conservation—have been introduced.

The early problems of water utilization were simple. The wants of a sparse population could be supplied without regard to economy of use. Frequently, a small part of the supply has been, or still is, sufficient to meet fairly well the needs of the dependent people; but population is growing at a tre-

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

mendous rate—in some countries at nearly a geometric ratio diminishing but slowly—and with this increase comes the insistent requirement, although not always clearly expressed, nor yet fully understood, that better provision be made by human agencies to get for mankind the greatest possible good out of Nature's bountiful supply of water.

Originally water, like air, was free to be taken by any one who had need of it. Because of the large supply as compared with the small demand the water right had no value. Because early settlement was of necessity at points where water could conveniently be obtained in the desired quantity no elaborate appliances were needed to modify natural conditions of occurrence and flow. As soon as permanency in the place of living was found to be desirable the advantage of stimulating plant growth by the artificial application of water was recognized. In some measure, first small and, later, larger quantities of water were brought under control, and service to the community was commenced.

It is self-evident that early works—whether early refers to the progress of civilization or whether in a more restricted sense to any particular region—must have been simple in their nature, easy of execution, and constructed to achieve an immediate purpose rather than to fit into the comprehensive plans of water utilization that later developed. It is only natural, therefore, that frequent occasion has arisen to find fault with the wasteful methods of control and use of water in those regions where the demand resulting from increasing population began to exceed the quantity that was conveniently accessible and obtainable. Correction of established customs and practices has not always been easy. The rights of the individual as against those of the State are not always clearly defined.

It seems fundamental that sooner or later the State—meaning any large political organization—must step in, and wherever demand for water exceeds the supply or where it is foreseen that such a condition is proximate, must assume control and define the features of the project for water conservation. The most obvious step to be taken in the case of the flowing stream is so to regulate its flow by storage and related works that it will render a maximum of service and do a minimum of damage.

In the United States, where the wise policy has prevailed of encouraging private enterprise, generous provision has been made for the early development of natural resources, including the use of water; and private rights have been established and are generally recognized, which do not, however, always comport, as they should, with the best interests of the country.

It is only in recent times that the privilege of utilizing parts of the public domain for water storage is coupled with conditions that make for the fair protection of public interests by limiting the granted right in the matter of time. The indirect control exercised by Federal authority over the waters of the lesser streams, however, frequently results in an invasion of the rights of the individual State. There is here sometimes conflict of authority that should receive timely attention. Just where, in other words, should Federal control stop and State control begin?

It seems that wherever a right of way, whether for conduit lines or for storage purposes, is required over lands of the public domain, the Federal Government has assumed a right of regulation and control which interferes in no small degree with the general conception of State control in such matters. There can hardly be any question that, when authority was first granted to the Secretary of the Interior to permit the use of lands in National parks and in forest reservations for the development of water resources, the functions of his Department were to be mainly ministerial. Such rights of way, it is fair to believe, should have been granted in all cases of merit and honesty of purpose, provided there was no material interference with the enjoyment by the public of the lands in park or forest for the purposes for which they were set apart. This view unfortunately has not prevailed. The Government conditions imposed in any grant of a right-of-way privilege may and frequently do supersede and annul the rights supposed to have been acquired under State law.

Attention may be directed in this connection to the so-called Raker Law granting certain reservoir rights in a forest reservation to the City of San Francisco, Calif. There is a conditional grant in this law for the utilization of a few acres of ground for water storage purposes made by Congress on the failure of the Department of the Interior to do its plain duty in the premises. No one can read this law without deploring the interference of Congress in matters that should never have been brought before it, nor yet without feeling that San Francisco's representatives were unwise in accepting the imposed conditions.

Where then shall the line be drawn between control by the State and control by the Federal Government of the use of water in the flowing stream? No complete reply to this question will be ventured at this time, but attention will be directed to only one or two matters connected therewith. It has already been stated that regulation of stream flow is essential to obtain for the community the maximum service from the water. Whenever a stream is an international or interstate stream the comprehensive plan of regulation and control should originate with the Federal Government. When, however, the stream originates and flows in one State only, there should be no interference by Federal authority save only in the cases in which navigable waters are affected by regulation. The devising of far-reaching plans by the Federal Government and by the several State Governments should be accepted as a fundamental duty, and after the adoption of any such plan, development whether by the Federal Government, by the State, or by private effort, should conform to the adopted program, and, of course, to subsequent modifications thereof.

It would be well, wherever practical, for the State to take charge of stream regulation, to build the necessary reservoirs, and to wholesale the output. The State of Massachusetts may be referred to in this connection. In that State authority has been given to a Water Board to issue State bonds to an amount which reached a highest limit of more than \$40 000 000. With funds realized from the sale of these bonds, storage works were constructed and main conduits were built. Water is delivered by them to Boston and to about twenty-

eight or more cities and towns near Boston. The delivery is wholesale, each community being free to arrange for the distribution of the water to the consumer as it sees fit. The ownership of the local distribution plant may be either public or private. As the interest and sinking fund requirements to meet the State's obligation under the bond issue have been fully met from the revenue resulting from the payments at wholesale for service rendered by the Water Board, a tax or assessment has never been levied by the State in this water matter. Those who get the service have paid the bills.

No reason is apparent why the example set by the State of Massachusetts should not be emulated by the Western States where regulation and full use of the water supply is of vastly greater importance, because of relative scarcity and lack of rain, than in the New England States. Recognition of this greater need for Government help prompted the passage of the Reclamation Law of 1902 by the United States. In this law, however (perhaps, of necessity), were embodied several provisions which, to say the least, were unfortunate. The appropriation for the use of the Reclamation Service was an indirect appropriation. The moneys realized from the sale of public lands were allowed to flow into the Reclamation Fund instead of going where they should go—into the General Fund of the United States Treasury. This gave to a number of the States a feeling that they were contributing inordinately to developments in other States. There should have been no reclamation law until the country was ready to make a budget allotment for a well-considered reclamation program.

Again, in another matter, the only additional point to which attention will be directed, the law set out to accomplish too much. Partial regulation of stream flow was to be combined with land settlement projects. No project could be undertaken unless coupled with the subdivision of large land holdings.

The Government, perhaps justified by the fact that it was giving a large bonus, in the shape of interest relinquishment to the land owner, assumed a large measure of direction as to the operation of the individual farms, although without material effect on the use of the land for speculative farming. Without expressing any opinion as to the timeliness of this Government entering on land settlement plans, it seems unfortunate that Government activity in the reclamation of land by irrigation was not restricted to the storage and wholesaling of water, leaving the construction of works for its distribution to the individual farm, private initiative, or district organizations, without any onerous settlement restrictions. It would seem wise now to take heed of the lessons of the past and to separate land settlement enterprises from the regulation of stream flow and the wholesaling of the water output.

F. H. NEWELL,* M. AM. Soc. C. E. (by letter).†—Dr. Widsøe has given a broad review of the situation, developing so many lines for discussion that it is necessary to choose among these, selecting preferably the one which is of most immediate interest to engineers and to economists concerned with the

* Pres., The Research Service, Inc., Washington, D. C.

† Received by the Secretary, August 3, 1925.

extension of irrigation development, namely, the use of public funds in reclamation.

The financing of irrigation development by private capital is covered by the other papers* and discussions presented with that of Dr. Widtsoe. These bring out certain advantages and difficulties, so that it is unnecessary to do more than refer to the fact that the great development of the country through irrigation always has been, and presumably always will be, through the wise use of individual and corporate funds, invested under State law. The Federal Government as such has relatively little concern with these private enterprises. In a broad way it may be said that, as pointed out by Dr. Widtsoe, irrigation was initiated by private enterprise and has been carried on with success; and it was only because of the existence of certain rather temporary conditions that the Federal Government was brought into the matter.

For every acre reclaimed by the use of Federal funds, as shown by the Census reports, ten acres or more have been irrigated by private enterprises. At present a larger amount of money is being invested in irrigation or reclamation district bonds and in other securities than is being appropriated by Congress. Therefore, the question may be properly asked as to why any Federal money has been or is being diverted for this purpose, also whether the reasons that called for the imposing on the overloaded taxpayers of the additional burdens of the costs of Federal irrigation should be continued.

Public Lands.—The argument for Federal reclamation of arid lands, as pointed out by Dr. Widtsoe, was based primarily on the fact that, at the time agitation was started for this Act, the Federal Government was the owner of more than 500 000 000 acres of land, a small proportion of which, possibly 1% or 2%, was so located and of such character that it might ultimately be utilized in home-making. At that time, also, about 1895-1901, the larger capitalistic enterprises, including those initiated under the Carey Act, for storing and distributing water to the arid lands were passing through a period of depression. They had not paid, and neither the State laws nor the prevailing practices provided adequate safeguards such as to render feasible further reclamation on a large scale.

There were at that time also not only enormous areas of lands which might be irrigated (some of it in ownership by the Federal Government), but a strong demand for these reclaimed lands. Population was still pushing westward and there were insistent demands for the extension of the principles of the Homestead Act. The easily tillable lands in Kansas and in other States of the Great Plains region had been taken up, but the tide of emigrants still continued to pour into and through these States, with the corresponding demand that in some way or another the Federal Government make available more tillable farms. The only way to do this appeared to be by providing water for some of the vast extent of arid land.

Moreover, at that time there still remained unappropriated considerable quantities of water flowing from the mountain areas, especially in the streams that crossed State lines. State laws and the decisions of the Courts were

* *Proceedings, Am. Soc. C. E.*, March, 1926, Papers and Discussions, pp. 396-433.

still in a somewhat chaotic condition, and it was shown that unless the Federal Government would step in, build some of the large storage works, and make far-reaching plans, it would not be practicable to utilize effectively the great resources latent in the proper control and use of these streams.

Under the urge of these conditions, especially the demand for more lands which might be used as homes for citizens, and because there was recognition of the need of comprehensive plans for development, active propaganda, liberally supported by the Western transcontinental railroads, was kept up to the point where the majority of Congress was convinced that here was a necessity and a duty. It may be pertinent to state that this education of the public, initiated thirty years ago, has been carried on so thoroughly and effectively as to plant deeply in the minds of the present generation the fact that, somehow, the Federal Government is committed to a policy of reclamation, even if the conditions that required it have largely disappeared.

In these past thirty years, especially in the last decade, there has been practically a reversal of the conditions on which arguments were based for Federal intervention in these local or State enterprises. First and foremost, it may be said that there are no longer any considerable body of reclaimable public lands. Of course, it is admitted that here and there are many isolated areas, usually inferior in character, which have been passed over or rejected again and again as being unavailable. This condition has been pointed out* and it was shown that the total extent of public land entered from 1901 to 1922, in the reclamation or public land States, aggregated 344 000 000 acres, leaving as unappropriated and unreserved, possibly open to homestead entry, not quite 190 000 000 acres, practically all of which is rough mountain lands, mainly of scenic value, having little vegetation, although susceptible of use occasionally in wet years as open or free grazing.

The greatest change, however, has been in the cessation of demand for irrigated or reclaimed lands, accompanying the general decline in agricultural population. Instead of a steady pressure for more homes on the land there is a tendency in the other direction. Thousands of families are going from the country to the city or to centers of industry where the earning power of the individual is far greater. Millions of acres already reclaimed by irrigation and drainage are awaiting settlers. The owners or managers of these reclaimed tracts are moving heaven and earth to get people who will live on the lands either as owners or tenants; they are offering every conceivable inducement to attract farmers.

The main question brought out by this presentation of the history and problems of irrigation development in the West is as to whether it is now wise and desirable to continue these efforts to reclaim more land and to attract more settlers. The answer involves the further question as to whether all taxpayers should be called on to spend more money, adding directly or indirectly to the present burdens of taxes—such as must follow from efforts of the Federal Government to get more settlers to occupy lands in the West.

Subsidy.—It is assumed that it is a proper function of the Federal Government to use its money and influence to bring about the most complete

* Reclamation Record, January, 1923.

development possible of the Western or public land States, even if most of the public lands have been converted into National Forests, or otherwise disposed of. This is a policy that has been accepted for a generation. Assuming, however, that this is correct, it is of great importance, to all engineers and economists, as well as to all citizens, to consider the limits to which the Federal Government, co-operating possibly with the State governments and local or district authorities, can and should go in offering subsidies or in giving bonuses to the land owners in the West, in order to secure the desired cultivation and use of their lands.

It may be urged in this connection that the Federal Government is not appropriating the money of the taxpayers but on the contrary is using special funds derived mainly from the sale of public lands. This, however, is merely "begging the issue", as Congress diverts from the Treasury to the Reclamation Fund certain sums which otherwise would go into the Treasury and which would relieve the taxpayers from a corresponding burden.

The original Reclamation Act, signed by President Roosevelt on July 17, 1902, set the limit of this Federal subsidy or bonus at ten years' exemption from payment of interest. These debtors to the Government for the water which made their lands valuable, were to be stimulated or encouraged to take up the Government land, freely given to them, or to utilize other lands purchased by them. It was believed that this exemption from interest payment, together with possible freedom from taxes on the unpatented public lands, would induce the settlement and development of these lands.

As a matter of fact, this ten years was in addition to a considerable lapse of time during which the reclamation works were being completed or "tuned up," so that even under this ten-year exemption many of the land owners enjoyed the use of the water for, say, fifteen years, and at relatively small expense.

In fact, all large reclamation enterprises, with a few notable exceptions, must be subsidized directly or indirectly. This is because the land when ultimately reclaimed and settled does not have an economic value equalling the cost of reclamation and other work put on it. These subsidies coming to the land owner may be concealed under many forms, but they can always be found by careful analysis of the total original cost and accrued interest. Much of this subsidy in a case of private enterprise is in the nature of failure of the original investors. The stockholders and even the bondholders of most of the large irrigation enterprises lost heavily, the works built by them as "involuntary philanthropists" were ultimately paid for at less than cost, this loss being practically the equivalent of a bonus or subsidy to the land owners.

Knowing this condition, the Roosevelt Reclamation Act was adopted under the belief that this ten-year exemption would be an adequate reward to the land owners. It is now generally believed that this is not adequate and the real question that confronts the engineer and economist, when stripped of the emotional, political, and other side issues, is simply "How large a subsidy should be paid to induce reclamation cultivation and settlement of waste lands? What are the limits that should be set?"

With ample funds and no limits set to further bonus or subsidy, it is possible to carry out any of the projects now proposed. If, however, it is decided

that the bonus or gifts to the land owners are to be limited, then only a few of the more desirable of the proposed Federal projects may be safely undertaken.

Infinite ingenuity and almost endless oratory has been used to obscure this simple fact. The politicians have succeeded in creating a smoke screen of sentiment, involving social and political considerations. Dust is thrown into the eyes of each investigator and every effort has been made to divert his attention from this, the real issue, namely, not what will be the first cost but what will be the ultimate contribution to the land owner on the project.

The benefits of reclamation, great in themselves, have been magnified or distorted. Sentimentality has run riot, distracting attention from the question which should be stated in the plainest language, namely, "How much must be donated by the taxpayers directly or indirectly and how does this bonus compare with the ultimate real value of its benefits to the land owners?"

One of the elements tending to obscure this question is the failure to recognize the fact that the liberality of the Government in giving this bonus or subsidy is not conferred on the public or on a large group of people but on a limited number of land owners. These owners may or may not be settlers, water users, or farmers. The reclamation works built at Government expense provide water only for certain designated tracts. It is only through the wise use of these particular pieces of land that benefits can come to the public at large. In a sense the whole question is in the control of these land owners.

In the case of subsidies granted by the Federal Government for hard roads, it is usually understood that these roads can be used by any one. In the case of irrigation works, the reverse is true—the use of the water must be narrowly limited to carefully defined areas. If the owners of these lands do not utilize the water to the best advantage the investment of Federal funds is a corresponding loss. To prevent such loss, it is now proposed to exercise a still larger or more drastic control of these land owners.

Moreover, it should be kept clearly in mind that the actual cultivator of the soil, the man who is trying to make a home and to develop the country, may not receive any benefit from the liberality of the Government. If he is a tenant or if he is trying to purchase a farm from one of the men who obtained it practically for nothing from the Government, his burdens may be greatly increased for the reason that the original land owner, or the speculator who has purchased the land, places an exorbitant price on it because of the assertion that the Government may not collect the debts.

Few students of the subject contend that the Federal Government should not subsidize reclamation in some way. This policy is now well established. It is almost a waste of time to discuss its wisdom. It is, however, of the greatest importance, for the continuation of reclamation, that the fundamental facts be faced clearly and that engineers and economists join with other citizens in the attempt to define the limits and the methods of the necessary subsidies. It will then be possible to consider in connection with each of the proposed reclamation projects whether under present conditions it will yield as large or as valuable results to the public as a whole as might come from the

investment of an equivalent sum of money in other or better considered undertakings.

In 1902, when the original Reclamation Act was passed, the Federal Government had an overflowing Treasury, it was not borrowing money to any considerable extent, and the taxpayers' burdens were moderate. It could well afford to let its surplus funds be invested without interest return for ten years in creating opportunities for homes or public lands as needed by its citizens. These conditions are now reversed.

Repudiation.—It was freely predicted in Congress and elsewhere when the Reclamation Act was under discussion that in one way or another the land owners who ultimately would become beneficiaries of this Act would endeavor to repudiate their debts to the Government in whole or in part. Enough was said to make the advocates of Federal reclamation peculiarly sensitive on this point and to cause vigorous protests of good faith; nevertheless, within 10 years, and before substantial repayments had been made, a vigorous and successful effort was made to extend the time of non-interest payment from 10 to 20 years. It was followed by other demands for extension of time of repayment, without interest, to 40 years, 60 years, or even to 90 years.

When the first Extension Act was under discussion in Congress it was urged by a few far-seeing engineers and economists that, although it might be within bounds of reason to permit 10 years to elapse without interest payment, yet after 10 or more nearly 15 years of settlement of a project, then the land owners could and should pay a small interest, $3\frac{1}{2}$ to 4%, or whatever the Government was paying at that time for the money, which, in a sense, was borrowed to replace the amount invested in these reclamation works. This suggestion was not accepted.

The success of the advocates of the Extension Act stimulated activities all along the line among the men who believed in whole or partial repudiation. They pointed out that the Government was expending millions of dollars, which it did not expect to recover, in the way of levees or protection works benefiting other agricultural lands; that it was spending hundreds of millions in hard roads; and that a single battleship built and destroyed without use cost more than a great irrigation reservoir or completed project. It was feelingly urged that the settler on the Government project had to endure all the hardships of pioneering and that, in a certain sense, he was performing a public function in aiding to reclaim lands and in making his home in a remote part of the country. For this, he should be rewarded by being excused from a part or possibly from the whole of the amount owed to the Government for the reclamation of his lands.

With every session of Congress these requests for further relief are pressed. The condition is such that although the immediate beneficiaries are in number an almost infinitesimal proportion of the farmers and land owners of the country, yet the petitions and cries for relief for these "wards of the Government" obstruct many other measures of importance to all citizens.

Limit of Subsidy.—This side of the picture is presented in order that consideration may be given to the limits which should be set for the time and amount of the subsidy or bonus to the owners of lands reclaimed by the

Government. Assuming it is a settled policy that such subsidy must be given, directly or indirectly, in order to induce a larger settlement of the Government projects, the lowest limit of this bonus or subsidy was that set by the original Reclamation Act as 10 years' freedom from interest. The Extension Act increased this to 20 years.

The estimate of this subsidy or cost to the taxpayers of the Nation has been discussed by officials of the U. S. Department of Agriculture. For example, on Salt River, Arizona* (probably the most successful irrigation project of the Government):

"The announced charge for water is \$90.00 per acre. Water was supplied to some farmers 11 years before payments began. In the beginning of the payments the accumulation of interest charges would have increased the charge to 187.84% of the announced charge, or to \$169.06; and at the end of the 20-year period, during which the \$90.00 will be repaid, without interest, the accumulated debt, with interest, will amount to 407.44% of the \$90.00 charge, or \$366.70. This represents the subsidy, per acre, to such a farmer in the Salt River Valley at the time when the Government will consider his debt discharged."

Enthusiasts may justify this expenditure and dwell on the benefits to the United States, but, while adorning these facts with flowers of rhetoric, they must not forget that behind the roses are the thorns of hard fact, namely, that the taxpayers of the country have had to pay, in real money, deftly taken from their pockets, this enormous sum. On less favored projects even larger amounts, as pointed out by the economists of the U. S. Department of Agriculture, must be paid if similar conditions are to continue. These facts should be presented in such a manner as not to alarm, but to stimulate action along lines which will bear the most careful scrutiny.

After facing squarely the fact that an enormous but partly concealed subsidy is being paid to a relatively small body of land-owners and weighing the advantage to these land owners and to the public, the people can consider on their merits the new enterprises which may be presented and which demand still larger subsidies. In such consideration should be kept in mind the fact that private or corporate capital undertaking reclamation enterprises does not ask or expect any large bonus or subsidy from the general taxpayers. Are the new projects or extensions of existing projects likely to be of sufficient value to the taxpayers of the country to justify their construction, taking into account the fact that if built by public capital they must be heavily subsidized, also that there is a strong and increasing sentiment toward repudiation under various guises? Congress must answer.

Land Owners.—Dr. Widtsoe has pointed out that in all this work, whether by private or public initiative, the man to be considered is the man on the land. As he states:†

"The heroic figure in irrigation development is the water user—the man and woman, who throughout the years live upon the land, till it, and make it yield enough for life's subsistence. Capitalist and engineer, statesman and tradesman, are all necessary, but compared with the water user are of secondary

* *Bulletin 1257*, U. S. Dept. of Agriculture, August 23, 1924, p. 12.

† *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 399.

rank in bringing irrigation development to success as measured by the standards of the age."

This is true; all consideration should be given to the real home makers. Nothing should be too good for the man and especially the pioneer woman; but it is proper to ask whether the plans and policies proposed, if carried out, will result in real good to this relatively small class of people, selected somewhat arbitrarily from the great mass of other pioneers and farmers. Does not the interference, although well intentioned, the persistent offers of advice, of loans of money, tend to reduce the energy and independence of these people and to get them deeper and deeper into debt from which escape, during the present generation, or even in the next generation, seems possible only through the door of repudiation, so temptingly held open by the local politicians?

In all these discussions of benefits which could or should be conferred by a generous Government on this small class of pioneers, there is a tendency to confuse two entirely distinct groups of individuals and to include under the name of "farmer" various kinds of people who may or may not be actual water users or residents on the land. The public appreciates that the farmer is at the base of civilization. Popular sentiment supports any measure that seems to be of benefit to him or that he thinks may help him. Everything he really needs must, of course, be given first consideration but, in the case of the Government projects, is it really the farmer who is most conspicuous in his demands? Not every land owner on a Government project is a farmer. The mere fact that a man owns one or more farms or farm units given to him or to his predecessor by the Government does not make him a real farmer.

The land owners on Government projects, who for the time are the immediate and direct beneficiaries of the liberality of the Government, theoretically may be called farmers. Through a strict enforcement of the law they should be actual residents and water users; but the fact has been brought out again and again that a large proportion of the persons owning lands on Government projects can hardly be called farmers in the sense that they are skilled or experienced in farming or expect to spend any considerable part of their lives on the farms, made tillable by the use of Government funds. These conditions have been discussed particularly in Congressional hearings,* which showed that these land owners represent every conceivable occupation running through the alphabet from actors, architects, and artists, through the "butchers, bakers, and candlestick makers", down to undertakers, watchmen, and yardmen of the local railroads. Some are absentee landlords. Tenantry has increased to upward of 40 per cent. Now the cry coming from every Federal project is: "For Heaven's sake give us less advice, less talk, but get us more and better tenants."

Dr. Widtsoe's summing up of the means of solving the irrigation problems are those which may well be applied to any capitalistic undertaking: First, faith in the enterprise; second, study of its needs; third, diffusion and better use of knowledge; fourth, a broader recognition of the values of the undertaking; and, fifth, training for leaders, with particular emphasis on

* Before Committee on Irrigation of Arid Lands, H. R., 67th Cong., S. 4187, January 24-25, 1923, p. 24.

economics and law. All must agree with these conclusions as thus broadly stated. In their practical application, however, they reach into the much debated questions of over-production, or rather of lack of provision of adequate transportation and marketing facilities. The consideration of these latter brings up the problem as far as the Federal Government is concerned as to the limit of these direct or indirect subsidies and of the extent to which it can properly or wisely interfere with private enterprise and go into business paralleling or rivaling that of corporations and individuals. Already a strong reaction has set in against the Government plunging into new schemes, because as yet no limits have been set to the subsidies that will be demanded. Moreover, it has not seemed possible to set any limits when once the original limit approved by President Roosevelt, that of a ten-year exemption from interest, has been exceeded.

State Co-Operation.—In their perplexity as to Federal duties and subsidies some of the members of Congress have taken refuge in the now fairly well established "fifty-fifty" rule. They frankly admit that they see no defense against the assaults of the subsidy seekers. In their despair they call for the help of the States. If the people of any State, its Legislature, and Governor, will go "fifty-fifty" with Uncle Sam, as in the case of hard roads, or will guarantee to do their part in any reclamation enterprise, then the Government can afford to go into it. As a matter of fact, however, privately they admit that there is little probability of any State doing this because the local people are too well informed to be willing to venture their hard-earned money on a Government reclamation scheme. They are perfectly willing that generous Uncle Sam should spend his money on their State but they are not venturesome enough to risk spending the money taken directly from the taxpayers of their own localities. Already they cry that the State and local taxation is too high and they cannot see why they should contribute money for the benefit of small groups of land owners, even through entering into a partnership with the Federal Government.

While engineers and economists may not go so far as to fully agree with the conclusions stated* by the U. S. Department of Agriculture, namely, that "there is no justification for a National subsidy for land reclamation", yet as citizens to whom the public may turn for advice, they should consider carefully and impartially the limits which may be set to such a subsidy, balancing the gains to the public by the development and use of waste lands against the cost or increased taxes to the public. The Federal Government, as before stated, is definitely committed to the general principle of subsidies given directly or indirectly to certain public needs, such as post offices, the building of post roads and other highways, the maintenance of navigation, and commerce between the States, as well as the building of levees in connection with such commerce, the main value of which, however, is in the protection of certain agricultural lands. It may well be argued that the providing of opportunities for homes on lands which are now waste and useless can and should be a matter of National consideration. All are agreed that plans should be prepared and funds made available within proper

* Bulletin 1257, U. S. Dept. of Agriculture.

limits for reclaiming these lands as rapidly as there may be an actual need for them in providing prosperous homes for citizens.

The question of greatest concern is not whether a subsidy or bonus to the land owner should be provided by the taxpayers of the country but whether this particular way of subsidizing a small group of land owners is the best or will produce the largest benefits to the great body of taxpayers. Reclamation works, unlike waterways or highways, are not State-wide nor susceptible of use by the public in general. They are strictly local, and the direct benefit from them must necessarily go to a few land owners and local industries.

Because of such conditions, it is urged that local or State co-operation and support should be forthcoming. If the Government is to spend, say, another \$150 000 000 in reclamation works to make available in the future more homes for citizens, it should be considered in the broadest possible way whether this \$150 000 000, or more, might not otherwise be used to produce larger results in home-making and benefit to all the taxpayers. Is it not a fact that lands can be reclaimed or made available for home makers throughout the United States at less cost and can be occupied and improved with less hardship and uncertainty than in the case of the new irrigation enterprises? Not only is this a fair question, but it is one which it is the duty of every broad-minded citizen to consider.

The history of the projects pointed out by Dr. Widdowson is full of instruction and warning against continuing as at present. It illustrates among other things that although the Federal Government is a good spender, it is about the worst possible collector. In fact, it is generally recognized that Uncle Sam cannot get back from citizen debtors any very large part of the money owed to him, whether this is for public lands or for seed grain. In discussing plans for Federal farm loans or other advances, it is asserted that there would be little hope of getting back all the money advanced for loans if the public was not well-informed of the fact that the money thus loaned did not belong to the Government but, in turn, was borrowed from people who must be repaid.

The plan which past experience shows may be most likely to succeed is that which utilizes the experience of the Federal Farm Loan Banks and similar institutions. Assuming that the Federal Government should subsidize the reclamation of waste lands, this subsidy may well be confined to the first ten years after settlement and be limited to loss of interest on the investment. At the end of ten years, when real property values have increased, the owners of the lands that have been reclaimed and cultivated during that time should then be required to give to the Government what may be considered a negotiable mortgage, for the repayment of the cost of reclamation, with a small interest charge—no more than that paid by the Government for the money it is borrowing—plus, say, a 1% installment on the principal invested.

The best way to carry out such a plan will doubtless be to have incorporated under State law in each locality a Reclamation District with power of making contracts and of levying taxes. The bonds of this District may

be received by the Federal Government in full satisfaction of all expenditures; at a suitable time these bonds may be put on the market and sold for what they will bring, thus closing up the entire transaction as far as the Government is concerned and relieving Congress from the annually recurring demands.

As a compensation for the benefits to the whole country, the Government may well assume the loss of ten years' interest or more on its investment and also the discount in selling the bonds. Careful consideration of such losses will show that they will be far less than under the present system. History shows also that the dealings between the District and its bondholders are likely to be conducted on fairer and more business-like terms than between a Government bureau, the local voters, and their political representatives.

The author has suggested many difficulties which for the benefit of those who have had a limited experience might well have been enlarged on, and further details given. In conversation with several friends, bankers and attorneys with experience in irrigation development, when the writer suggested financing by private capital, the consensus of opinion was suggestive of the backwoodsmen who seek a homestead for the first time, and on the back of the pack leaving away remarks, "They ain't no such animal." Just as the backwoodsmen was wrong, so were these financiers wrong. The backwoodsmen particularly tried to avoid expense, to require a minimum of water and an attendant who understands his mode, his limitations, requirements, and necessities. The same applies with equal force to financing an irrigation project and particularly financing by private capital. The writer was struck first by the statement in reviewing the paper the writer was struck first by the statement in connection with the desirable reclamation of arid lands wherever possible that "it should be done at such times and in such ways as will fit in with the changing needs of the country."

That there is now under cultivation in the United States a sufficient area to meet all present needs is demonstrated by the presence of a surplus of nearly every agricultural product, even in years of low production, by the prices obtained by the producer, and by the marked shift from rural to urban life. No "back to the farm" propaganda—no "stay on the land" slogan—will avail so long as the farm is unprofitable or so long as there is the present

THE discussion for the paper by Mr. Shepard, read, presented at the National Irrigation Conference, July 2, 1925, and published in March, 1926, Proceedings of the National Irrigation Conference, in order that the views expressed may be brought before all members for further discussion. Mr. Shepard's statement and the statement of the other members of the conference are published in the same issue of the Proceedings.

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THE FINANCING OF IRRIGATION DEVELOPMENTS BY
PRIVATE CAPITAL

Discussion*

By JOHN E. FIELD, M. Am. Soc. C. E.

JOHN E. FIELD,† M. Am. Soc. C. E. (by letter)‡.—The author has selected for his subject the most vital and all-controlling problem of irrigation development—that of financing. To finance a project necessitates that it be sound in every respect, regardless of the method or of the source from which the funds must come. To be sound financially requires that the interest and principal be paid regularly and surely and that, at the allotted time of maturity of the obligation, the principal be paid.

The author has suggested many difficulties which for the benefit of those who have had a limited experience might well have been enlarged on and further details given.

In conversation with several friends, financiers and attorneys with experience in irrigation development, when the writer suggested financing by private capital, the consensus of opinion was suggestive of the backwoodsman who, seeing a dromedary for the first time, studied it carefully and on finally moving away remarked, "They ain't no such animal." Just as the backwoodsman was wrong, so were these financiers wrong. The dromedary is particularly fitted to arid regions, he requires a modicum of water and an attendant who understands his moods, his limitations, requirements, and possibilities. The same applies with equal force to financing an irrigation project, and particularly financing by private capital.

In reviewing the paper the writer was struck first by the statement,§ in connection with the desirable reclamation of arid lands wherever possible, that "it should be done at such times and in such areas as will fit in with the changing needs of the country."

That there is now under cultivation in the United States a sufficient area to meet all present needs is demonstrated by the presence of a surplus of nearly every agricultural product, even in years of low production, by the prices obtained by the producer, and by the marked drift from rural to urban life. No "back to the farm" propaganda—no "stay on the farm" slogan—will avail so long as the farm is unprofitable, or so long as there is the present

* This discussion (of the paper by R. E. Shepherd, Esq., presented at the Summer Meeting, Salt Lake City, Utah, July 8, 1925, and published in March, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Denver, Colo.

‡ Received by the Secretary, August 1, 1925.

§ *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 404.

marked difference in comforts, luxuries, hours of labor, and net earnings between the farmer and the mechanic. When a man with \$12.50 can buy an automobile, buy a home "just like rent", and clothe himself and wife in silk on the installment plan, the acquisition of a farm on the amortization plan will not appeal to the young farmer unless he sees, in addition, a definite promise of profit and financial independence within a reasonable time.

The Bureau of Economics of the U. S. Department of Agriculture states that there is now under improvement in the United States an area which, if properly cultivated, would meet the needs of the population, with due regard to the probable increase, for the next fifty years. Whether or not this estimate is correct, it is certain that there is no present or crying need for more land. Twenty years ago there was a slogan, "Land for the landless man and men for the manless land". Now this slogan must be discarded in both its phases. To-day there is much land in the arid West under canal, with good water supply, markets, railroads, and other necessities crying for men. Farms are being offered at less than the cost of the irrigation works. It seems, therefore, that to discuss financing the development of new irrigation projects is a waste of time, or at best an academic question. Not so, however, with the financing of existing projects, of finishing and of perfecting existing systems. Irrigation engineers of the older generation take something of a morbid pleasure in analyzing their failures and mistakes.

There is a time for all things, and the author in limiting development to the changing needs of the country has struck the keynote. He asks to what extent is it prudent to embark with private enterprise and capital in the work of land reclamation. In answer to that question the writer would remind the older men and inform the younger ones that back in the early Nineties the idea fostered by the Irrigation Congresses was that the Government should build reservoirs to supplement the water supply of existing systems, that this and this only was to be the program. How far has been the departure from that idea and what troubles and failures that departure has wrought are too well known to require recital here. Next came the suggestion that main canals covering public lands should be built. This development was probably wise, but still no hint, even at the time of the passage of the Reclamation Act, had been given that laterals would be built, much less that the systems would be operated and maintained by the Reclamation Service, nor was there any suggestion that the Service would build drainage systems.

As the Project Engineer of the North Platte Project the writer opposed the building, first of any laterals, then of any except those involving great difficulty and expense, and never as long as he remained in the Service did he countenance the idea of Government maintenance and operation. Referring specifically to that project, one can easily imagine the limiting of Government activity to the construction of the Pathfinder Reservoir.

Private capital was ready and anxious to build the canals, and indeed it was necessary to take measures to prevent private enterprise from building both the Interstate and Fort Laramie Canals. The Tri-State Canal, covering much of the land under the Interstate Canal, was actually built by private capital, and now irrigates land originally embraced in the area intended to

be served by the Government Interstate Canal. The first mistake was made in extending the Government work to include the lateral systems, but the great and serious mistake was made when the Service undertook the work of maintenance and operation.

The story goes that two travelers down in Arkansas, driving along a road through the woods, were startled by seeing a wild-eyed woman with streaming hair break out of the underbrush into the road and then, like a startled deer, plunge into the opposite thicket. Hard on her heels came a man with scraggly beard and disreputable appearance. He inquired of the strangers if they had seen a woman cross the road. They denied that they had and asked what he was doing, and he replied, "She's my mammy and she's trying to wean me." The way the settlers are acting and the way the Reclamation Service has treated them in the past reminds one of that story. Mr. Mead and Secretary Work evidently have concluded it is weaning time.

Answering the question specifically, the co-operation of the Government should be limited either to the construction of reservoirs for supplemental supply, or if it is deemed best that this Government become paternalistic, socialistic, or communistic, then let it go ahead and do everything of every kind and nature. Personally, the writer much prefers that American initiative and ingenuity be preserved and developed, even if some hardship is entailed. The lack of responsibilities imposed, the absence of sure and certain penalties for shiftlessness and procrastination, is responsible for much of the trouble on irrigation projects. Most of the trouble is peculiar, not to Government projects, nor to irrigation projects, nor to farming in the arid West, but to the condition of farming itself.

The author has listed nine questions* all of which must be answered in the affirmative if an irrigation project can be pronounced sound. The first three deal with the land; the fourth and fifth with the water supply; the sixth and seventh with markets; and the eighth and ninth with the cost of the project and revenue from the land. All irrigation engineers worthy of the name have in the past thirty years considered carefully in their reports the first seven and many have considered the last two. Unfortunately, some engineers have considered it outside their province to pass on the financial aspects of a project, leaving that matter to the promoter, the financier, and the colonization agent. Some have even limited their activities to the cost of construction, avoiding the vital matter of character of soil, climate, products, water supply, and markets to some one else. In this, the engineer failed to grasp an opportunity to do much useful, valuable constructive work, and to rise above the level of a surveyor, and of a "slip-stick artist". In consequence, many of the failures are chargeable to the engineer.

With the nine conditions imposed by the author the writer is in accord, but he would enlarge the list. The value of a farm is divided among the land, the water, the improvements, and the expense of making it a going concern. These four elements on the public land with ordinary costs would be: Land, \$1.25; water, \$35; improvements on 80 acres, \$4 000, or \$50 per acre; and five years to make it a going concern which might be put at \$50 per acre;

* *Proceedings, Am. Soc. C. E., March, 1926, Papers and Discussions, pp. 405-406.*

total, \$136.25. There must be added to this figure some bonus or unearned increment, in order to induce the farmer to break off his old association and to suffer the hardships of the pioneer and settler; he must believe that within a reasonable time he will be considerably better off by making the change. It would seem that at the end of 10 years his 80 acres should be worth \$200 per acre. However, as the author states, "the unearned increment was guessed at and quickly capitalized." The final holder paid the following:

To the original entryman.....	\$63.25
Improvements.....	50.00
	<hr/>
	\$113.25
Interest on \$113.25 for 5 years at 6%.....	33.97
Going concern.....	50.00
Water.....	35.00
	<hr/>
Total.....	\$232.22

This is \$32.22 more than the land fully developed was worth. At first, a profit is taken by the holder of the land when he sells, but later a loss is taken, of necessity, by the settler and the bondholder. These figures indicate that the cost of the water should not greatly exceed \$35 for land worth \$200 when fully developed.

The history of an Irrigation District bond is interesting and informative—taken by the broker or contractor for 60% of its par value, sold by the broker at par and the profit divided between the broker and the contractor. Interest is paid for a few years from the proceeds of the sale of bonds, and then the broker is no longer interested. The interest becomes delinquent due to lack of settlement and the fact that the settler paid out all his available resources in the purchase of a relinquishment for land, and for improvements, if any, and in living expenses. The bonds drop to 60% of par. A new series of bonds is issued and exchanged for the old at 60 per cent. Thus, the project becomes a success, and the settler gets his water at cost. Often the exchange of old bonds for new ones at less than 60% of par is effected and the process of refinancing is often repeated until the indebtedness is reduced to a point which the settler can pay. The Government is in effect a bondholder. Will it, too, take a loss just as all others have done before?

So much for the past; now for the future. The choice is between Socialism and Government aid or private financing and independence. Assume that the nation is not yet ready for Socialism, then how shall private financing be accomplished for the purpose of completing present projects and for building those which may be found feasible as time goes on? The author proposes a form of Government corporation similar to the Federal Land Banks, to finance the reclamation of waste lands, whether privately owned, State owned, or in the public domain. His suggestion is a good one. There being no need, however, of adding to the total farm area, the business of such a corporation should be limited to taking over, completing, and liquidating existing Government projects. To give it greater scope would

be to perpetuate the scramble for public money, to inject politics into its operation, and to subject the corporation to importunities and influences, as was the Reclamation Service. This would lead to similar unwise, unnecessary undertakings. If its activities were so limited and if it proved a success, then, with the experience gained and with the possible development, say, in 25 or 30 years, of a healthy demand for more farms and for the development of areas and projects impossible to private enterprise, a new corporation could be created; but the salvaging of existing and the undertaking of new projects should be separate and distinct. It is to be anticipated that new work would be so much more attractive that the salvaging of the old projects would be neglected and become secondary.

Having been the salvaging engineer on many projects in the last fifteen years, the writer knows how hard that work is, how unsatisfactory, how devoid of glory, and how poor the remuneration. As compared with the enthusiasm, the publicity, and the advertising on new projects, such work would be like the "cold gray dawn of the morning after." To improve and complete projects where the land is principally in private ownership the landholders should give individual mortgages on their land and raise the necessary funds. If the land is already encumbered either by mortgage or Irrigation District indebtedness, the creditors must join the owner of the fee title in the new mortgage. There is no reason for relieving at public expense either the land owner or the mortgagee of the results of his mistakes. Government aid should be confined to:

First.—The gathering of data and research work necessary to a proper analysis of proposed irrigation projects.

Second.—The construction of reservoirs for supplemental water supply for existing projects.

Third.—The completion of projects already under way on which the future expenditure is less than the value of the enterprise, and which, when completed, can be bonded and the bonds sold at par and at not more than 6% interest, thus definitely getting rid of Government control and imposing the burden of responsibility where it belongs, on those most vitally interested—the water users.

The Government evidently hopes to induce the States to take over its projects. Such a step would simply be "jumping from the frying pan into the fire." When a change is made it should be from the Government to the water users' own organization. Corporation control and management is bad; State control and management would be disastrous. Government control has had a trial over two decades and is in a mess, to say the least, and, as far as successful financing is concerned, could not well be worse.

The engineer should pass on the nine conditions mentioned by the author and, in addition, show the cost of improvements, going concern, value, and add an estimated bonus or unearned increment; if colonization is necessary add 25% for that item. If these items aggregate less than the value of land already developed, then the project could go ahead. However, if expense for colonization is necessary, the greatest care should be taken to verify all other

costs. Quick colonization and development are essential to success. More good projects have failed through lack of proper and quick colonization than from all other causes. A project simply cannot stand years of accumulating interest charges and operation and maintenance costs.

The writer's recommendation to those contemplating irrigation construction at this time is like that of Punch to those contemplating matrimony, namely, "don't"; but, like Punch, he does not expect to have his advice taken, nor would he wish it to be unless his own money were involved, for on the dreams of the optimist and his courage is progress founded. The optimist and the promoter and those of whom it is said there is one born every minute, will keep development from ten to twenty years ahead of its time. It can no more be stopped than can matrimony on the part of those whose heart is in it. It is possible, however, to "give them a run for their money," and to see to it that the projects are sound, although possibly premature.

The criticism of all proposed developments and the methods suggested is that a false stimulant is almost invariably proposed in the form of money without interest, long-time payments, houses furnished, land plowed and planted, livestock sold on credit, in fact, a proposition which any "ne'er do well" will accept, the shiftless will try, and the incompetent will undertake. It simply makes more difficult the ultimate and necessary weeding out of the undesirables; it puts the sincere, earnest, and competent in competition with the notionate itinerant, the man who has failed in everything else, who is willing to try something where there is nothing to lose and everything to gain, especially when there is a prospect of selling out and moving on, leaving extra burdens on his successor, the burdens of increased cost, of discredited effort, and of disillusion and disappointment to overcome.

If this false stimulant is necessary, then the project is dangerous. If there is no demand—no insistent demand—for land and farm and home, then the time is not opportune and the project should wait. In these circumstances, any project that cannot secure private capital at a reasonable rate of interest is unsound financially. Non-taxable municipal securities are bringing $3\frac{1}{2}$ to 4% interest; the tax-free bonds of an irrigation district will find ready market if the interest payments are secure and certain. Let the saleability of the bonds be the final test, and let the policy be not to sell bonds below par, nor to give bonds for construction, not to employ any colonization agent, and to offer only reasonable inducements; then it will be certain that no unwise promotion will be attempted.

Furthermore, let each individual land owner assume a definite obligation and a personal risk and responsibility; let him know that the debt to the last farthing must be forthcoming; let him join the enterprise calmly, soberly, and in the fear of failure; let him know and feel that success depends on his personal effort and industry. Then responsibility will develop his judgment and his ability to meet and overcome difficulties. Those projects already started should be classified and those which require an expenditure greater than their value should be definitely abandoned until they can pass the acid test here proposed.

Westerners wish the West to grow and to prosper, they are impatient, they are selfish and care little whether these developments hurt other sections, they believe in the "survival of the fittest," in the immutable laws of Nature, of finance, and of political economy, and they believe in every project standing on its own merits. If they cannot grow as fast as they wish without a stimulant then they are willing to wait, preferring to grow slowly, surely, and safely rather than by alternate booms and depressions. They want no settlers who have to be bribed to come and live among them.

Let Westerners pray for patience and work for sound progress; pray as if there was no help on earth or from Washington; work as if there was no help from Heaven. Then they will return in the spirit to that time when they fought their way against desert and drouth and redskin and remoteness, fought with hope in ultimate victory and confidence in their own efforts and capabilities, when Uncle Sam was more of a myth than a reality, or rather when they themselves were a helpful part of Uncle Sam in all his majesty, power, and glory.

The criticism of all proposed developments and the making of money without interest, four-time payments, houses furnished, much food and clothing, livestock sold on credit, in fact a proposition which says "we'll do it for you" will never, the critics will try, and the important will undertake it simply makes more difficult the ultimate and necessary working out of the project. It puts the sincere, earnest, and competent in competition with the dishonest, the man who has failed in everything else, who is willing to try something where there is nothing to lose and everything to gain, especially when there is a prospect of getting out and moving on, leaving extra burdens on his successor, the burdens of increased cost, of discredited effort, and of disillusion and disappointment to overcome.

If this false stimulant is necessary, then the project is dangerous. If there is no demand—no instant demand—for land and farm and home, then the time is not opportune and the project should wait. In these circumstances any project that cannot secure private capital at a reasonable rate of interest is doomed financially. Non-taxable municipal securities are obtaining at 4 to 4 1/2 percent; the tax-free bonds of an irrigation district will find ready market if the interest payments are secure and certain. Let the solvability of the bonds be the final test, and let the policy be not to sell bonds below par, nor to give bonds for construction, not to employ any colonization agent and to offer only reasonable inducements; then it will be certain that no unwise project will be attempted.

Furthermore, let each individual and owner assume a definite obligation and a personal risk and responsibility; let him know that the debt to the fact is his; let him join the enterprise calmly, soberly, and in the face of failure; let him know and feel that success depends on his personal effort and courage. Then responsibility will develop his judgment and his ability to meet and overcome difficulties. These projects already started should be abandoned and those which require expenditures greater than their value should be definitely abandoned until they can pay the said interest and principal on bonds and loans.

PRESENT POLICY OF THE UNITED STATES BUREAU OF RECLAMATION REGARDING LAND SETTLEMENT

Discussion*

By MESSRS. B. A. ETOHEVERRY AND THOMAS H. MEANS.

B. A. ETOHEVERRY,† M. Am. Soc. C. E.—There is no doubt that the financial results of Federal Reclamation as far as reimbursing the Government is concerned, have not been successful. The Fact Finding Commission found that \$18 861 146 will never be recovered and that there will be a probable loss of an additional \$8 830 000. The total of these amounts is 20% of the money spent by the Government and includes no interest. It also assumes that payments will be made on the remainder, without additional relief measures, which past history would indicate to be a rather optimistic assumption unless a different policy for repayment is to be enforced by the authorities.

The failure in repayment has been largely attributed to the inability of the land owners to pay; and this has usually been charged to (1) the cost of construction, exceeding the original estimates of cost; (2) the inflation of land prices, or the excessive prices the settlers have had to pay for lands held in private ownership; and (3) the inexperience of the settlers and their inability to finance themselves. These conditions are in general not more unfavorable than those found on irrigation projects built by irrigation districts and other agencies. On the contrary, the land owners on the Federal projects have had the great advantage of having no interest to pay on their obligations and no cost of financing, such as discount on bonds. In addition, through the Extension Act of 1914 and several leniency acts, they have had a part of their payments due on the capital and on operation and maintenance deferred.

Critics have stated that some of the engineers of the Reclamation Service have been more concerned in building monuments of engineering than in the economic problems of successful land development. Even if this were true the water users have generally had more adequate water supplies and have demanded better service than on projects built by other agencies. In many irrigation districts the land owners have had to be satisfied with only a partial season's supply during the early period of settlement and development, up to the time when they could finance the storage works necessary to develop a full water supply. In other words, they have built at first what they could afford and not what they desired.

* This discussion (of the paper by Elwood Mead, M. Am. Soc. C. E., presented at the Summer Meeting, Salt Lake City, Utah, on July 8, 1925, and published in March, 1926, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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This was the economic solution when these Districts, before they had become established, had to discount their bonds 10 or 15% or more, and pay interest at 6 or 7% or more. It is true that many failed, but on the other hand many were successful and the failures would have been much decreased had the projects been financed with no interest and no discount on bonds. The past policy for repayment to the Federal Government has been largely adjusted to fit the land owners who are either incompetent, unsuccessful, or unwilling to meet their obligations to the Government, with the result that the more successful and those who were able to pay and, in many cases, willing to do so, have either accepted the advantages of blanket reliefs or have made their payments because of their sense of pride in meeting just obligations. That non-payment has often been due to unwillingness to pay is indicated by the records of the Reclamation Bureau.

The Commissioner of the Bureau states* that in 1924 one of the good projects paid only 7% of what it cost to operate it; another paid only 15 per cent. One project that owed \$440 000, only paid \$25 000, or 6%; another project that owed \$16 000, paid only \$69; and another that owed \$112 000, paid only \$6 000. The records show that many of the larger good projects received water for 11 to 14 years before any payments were made on the construction charges, whereas on other less desirable projects payments had to be made immediately after water was first applied. It is difficult to find any good reason for these inconsistencies in repayment. On one of the largest and best projects, enjoying a large income from hydro-electric power, water was first supplied in 1907 and the first payment on construction was reported in 1918. The net construction cost on June 30, 1923, was more than \$10 000 000 and the amount paid by the water users on construction to that date was less than 10 per cent. The economic conditions on this project are as favorable, or even more so, than on many irrigation districts where the land owners have had to pay in discounts and interest an amount nearly equal, if not equal, to the entire cost of construction.

Other projects have apparently showed a greater willingness to pay or have not taken full advantage of the leniency of the Government. The Orland Project in California is reported as the only one of the twenty-five Federal projects that has made without deferment its regular payments to the Government for annual costs and for operation and maintenance. Although the land owners suffered a 60% loss in crop production in 1924, due to the extreme shortage in water, they have maintained their record of full payment. There is no doubt that on some of the projects leniency was necessary because of economic conditions making the projects more or less unfeasible. For these projects it would seem that the best policy is for the Government to recover only that part of the cost represented by its useful value to the land owners. In the consideration of a new policy for Federal Reclamation the following fundamental questions arise:

(1) What shall the Government do with its existing projects?

(2) Should the Government undertake new projects for bringing new lands under irrigation?

* New Reclamation Era, June, 1925.

(3) If the Federal Government is to continue to use its funds for reclaiming lands, how can it be used to the best advantage, with reasonable assurance of its repayment?

The answer to the first question is that the fair value of each project to the water users should be determined and the project turned over to an organization of the water users which will give as good guaranties as possible for the repayment. This, apparently, is in accordance with the new policy of the Federal Government.

The answer to the second question is that, for the present at least, there is no real demand for such new projects. From the standpoint of the Federal Government there is no profit and from the standpoint of the settlers there should be no demand, if the complaint arising from many of them properly represents true conditions. Disregarding these complaints, however, there is now much land under irrigation which only lacks settlers, and in some localities there are lands fully developed which can be purchased at a cost less than that of creating newly developed farms. Mr. Mead has stated that on the five new projects authorized by the last Congress each settler will face an expenditure of \$200 to \$250 per acre before he has grown a crop or has established the productive value of land and water in irrigation. In the Annual Report of the Commissioner of Reclamation of June, 1924, the statement is made that well-improved irrigated land, fenced, equipped with buildings and growing crops, can be purchased in many of the Federal and private projects of the Northwest for \$150 per acre, including a paid-up water right.

With this condition existing there is apparently no general need of bringing new lands under irrigation for new settlers, to be financed by the Government or the State. New development should be postponed until the agricultural demands make it necessary. In some localities agricultural and economic conditions are favorable to success and there is now, or soon will be, demand for additional lands that will justify the undertaking of new projects.

The answer to the third question is that the Federal Government should not limit the use of the Reclamation fund by favoring a relatively small number of land owners at a prohibitive cost to the Government, especially when it apparently has resulted in creating a number of dissatisfied citizens. It should extend its help to the most meritorious projects throughout the West and adopt a plan or policy which will give reasonable security for, or even assurance of, repayment to the Government.

The sentiment has been expressed by some settlers that the Reclamation fund does not belong to the United States, but to the West. Many will differ with this view, but even if it is admitted as correct the West does not consist entirely of the land owners on a few projects. It certainly would be much more equitable to apply the fund to all the land development projects or to as many as possible.

It is well known that important causes of failures are the high cost of financing new projects and the large development cost due to the long period of time necessary to approach substantial settlement or development, especially on large projects. As previously stated, new irrigation districts are financed by discounting their bonds 10 or as much as 15% and they must pay interest

on the bonds at 6 or 7% from the time the project is started. Operation and maintenance costs must also be met. Later, the payments to retire the bonds must be started. The burden is excessive especially during the first ten years, or even longer for large projects, because the area brought under irrigation increases slowly.

The largest results should be obtained by using the Reclamation fund in accordance with the following policy.

The Federal Government should take the bonds of the Districts and hold them, either for a fixed period of ten years or more, or until the projects have been established to the extent reasonably required to meet future obligations, when it should sell them to reimburse itself. These bonds should bear a low rate of interest or even no interest (if the Government is to continue this practice) during the period that the Government held them. Should a project be only partly successful, or not economical, or not feasible because of engineering, agricultural, or economic mistakes, beyond what may reasonably occur on any project, the Federal Government should absorb the loss, cancelling some of the bonds so as to sell only the amount representing a fair value of the project.

The Government could protect itself by extending this aid only to projects which it considers sound, and in which it can obtain satisfactory guaranties against land speculation during at least the period that it held the bonds.

This plan could be applied not only to irrigation projects built by irrigation districts or other agencies, but to reclamation districts such as those in California where the cost of reclamation for levee protection, drainage, and irrigation has placed greater burdens relatively on the land owners than on the average irrigation projects. Many of these reclamation districts have had to be financed with 15% discount on warrants and 7% interest, followed by a discount on bonds.

It may be urged that Federal aid to the amount suggested will extend over a larger area than if it is applied to the more intensive aid required to build a project and also finance the settler, and therefore may not be sufficient to produce success. The fact remains that there are many irrigation districts where the land owners have had to carry much heavier burdens than on the Federal projects without deferments on payments and have generally done so with less complaints. This has resulted in the weeding out of the more incompetent so that the remaining land owners are good citizens who have retained their pride and sense of obligations. The larger and more difficult projects yet remain to be constructed; although they should not be constructed until the demand for agricultural lands has exhausted the supply of available good lands already under irrigation and awaiting settlers, they will have to be constructed, some of them in the near future, and Federal aid during the period of settlement will be most needed.

THOMAS H. MEANS,* M. A. M. Soc. C. E. (by letter).†—Mr. Mead's frank statement of the settlement problem confronting the Reclamation Service brings out sharply the difference between economic conditions in the early

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† Received by the Secretary, July 8, 1925.

days of Government irrigation and present-day conditions. The idea then prevailing was that all the Government needed to do was to build the reservoirs, or where reservoirs were not needed, to construct the main canals. Everything else could be done by the farmer's own labor. The reason back of this opinion was that the part of the works requiring any particular skill or necessitating the expenditure of money in large amounts, could not be well done by the farmer, but the lateral canals and the preparation of the land could be cared for by farm labor, thus placing the land on a producing basis.

In those days, when a farmer estimated the cost of bringing a farm into cultivation, he considered the cash outlay necessary to do the work; his labor and that of his family were not considered as a part of the cost. Ditch building could be done by his own labor and that of his teams. The provender for the team was raised at home with little cash required. Purchases of lumber and hardware in small quantities were the only cash outlays. Groceries and clothing for the family were the principal items of expenditure and in those days included very few articles. Vegetables, potatoes, beans, and other principal items of food were home-raised—the first effort was toward the raising of food for the family and the animals on the place. Cows produced the milk, butter, and cheese supply and the farmer knew how to slaughter a calf or a pig. The art of home-curing of meats was understood and the wife usually raised chickens and looked after the production of eggs and of many other things now bought from the store.

Nowadays all is different. The prospective farmer inspects the site of his new farm in an automobile. This vehicle consumes gasoline, oil, and tires, none of which can be produced on the farm but must be purchased for cash. The annual outlay necessary to keep a \$1000 automobile in running order to-day would have paid all the cash expense of an 80-acre farm twenty years ago. The upkeep of a Ford at the present time often exceeds the water bills and taxes on a quarter-section of land. Other habits of the present-day farmer require much cash. A diet of bacon and beans and home-made bread does not sustain life as it once did, and the farmer's table to-day is supplied from the ends of the earth. Many food products once thought to be luxuries are now considered essential to life and happiness.

A great deal of the difficulty of the present-day farmer and his inability to make good on the farm without a large amount of money comes directly from this fact. There does not seem to be any way of getting around the matter; automobiles, radio, movies, canned fruits, and foods from distant sources are necessities nowadays and no one seems willing to do without them. The fact is that more cash is required now than twenty years ago. The standard of living is higher. The day of the pioneer is gone. Few men now seem inclined to go out in the desert and carry on the hard work necessary to make a producing farm out of sand and water. Not only is the inclination lacking, but the ability to perform such a feat is possessed by fewer people. So far as any large percentage of American stock is concerned, it is poor material to use in conquering the desert. About the only recourse is to hand-pick men of special training for settlers and coddle them until they can be safely left to stand alone.

In addition to Mr. Mead's paper, the writer has had the opportunity of seeing some of the reports of the technical committees of the Bureau of Reclamation on unfinished projects and new proposals for extensions of projects. In none of these proposed new works is the cost of irrigation works less than \$100 per acre and in some cases it goes to nearly \$150 per acre. This cost is only for irrigation works, to which the farmer must add the cost of improvements and land preparation on the farm. Mr. Mead estimates the minimum expenditure to be from \$4 000 to \$10 000 per farm, or roughly, \$100 per acre. The total cost of the farm will then exceed \$200 per acre without interest. The particular projects referred to are in Washington, Oregon, and Nevada, in sagebrush regions. In addition to all this, roads and schools and all the other facilities for community life must be provided.

These statements emphasize the fact that the prospective farmer on one of these projects must be a man with considerable means. Men who have had experience with several settlement projects say they have never seen any considerable number of settlers possessed of nearly the amount of money required. It is incredible that many men with \$10 000, or even \$4 000, in cash, would want to undertake the hard labor, the privations, which even in these days cannot be entirely eliminated, and the uncertainty of the final outcome of the attempt to make a home out of a tract of desert land. A man who has had the ability to earn and save \$10 000 before he becomes old and decrepit seldom wants to attempt any such enterprise. The man who possesses \$10 000 which he did not earn usually thinks too much of the "bright lights" and the creature comforts to undertake to reclaim a desert farm. Among men with sufficient money the choice is limited, therefore, to a small number of country life enthusiasts who truly believe that the country is the best place in which to live and raise a family. There are still a few people of this sort who have enough money to reclaim a farm, but there are not enough of them to fill up a very large tract of desert land.

On the other hand there are a large number of men and women with small means, but with energy, enthusiasm, ability, and love of country life, who would be willing to wager a few years of hard work if there was a chance of winning a little money. Few people are willing to spend half a lifetime in paying for a farm unless it is possible to sell out at a profit. The farmer who spends \$200 per acre or more in making a farm out of raw sagebrush land in Nevada, Washington, or Oregon, is paying all the farm is worth. He is taking much risk and buying the land at its full value. It is not an attractive undertaking, but it does offer a method of acquiring a farm home that will enable many men to succeed. If the prospective settler has the ability to make a desert farm productive and has the hope of selling the farm for more than it has cost him, he will work, and work hard. The only hope in the future in reclaiming the arid West lies in the farmer with the hope of making money as the driving force behind him.

There are in America few men so constituted mentally as to be happy when struggling to pay off a debt which stretches out thirty years or more in the future—few who are interested in working a lifetime to earn a farm home. The hope of reward in the not too distant future cannot be eliminated

entirely; something must be done to make it possible for the farmer to pay up in a comparatively short time.

There are two problems—either to reduce the cost of reclamation or to subsidize the new settler in some way. There seems to be little hope of reducing the cost of irrigation works. All the cheap irrigation projects have been built. Construction costs are not likely to decrease much during the present generation. The farmer's expenditures on the ranch are more important than the construction costs and the tendency is toward the need of more cash rather than less.

Direct subsidy of reclamation farmers, beyond the freedom from interest, has not been seriously considered, but many efforts have recently been made to connect the Reclamation Service to enterprises in which direct subsidies would finally have to be made. On such projects as those where construction and farm costs equal or exceed \$200 per acre, there is no possibility of paying for the farm out of proceeds in any reasonable time. If reclamation is to continue the Government is faced with the necessity of selling water rights or farms at their agricultural value and not for the cost of reclamation works. It would be better for every one if, on such a project as the Spanish Spring Project in Nevada, the water rights were sold at \$50 per acre and the payments collected in 20 years, rather than have the full cost of \$100 or more per acre spread over 50 or more years. Raw sagebrush land is seldom attractive at costs of more than \$50. Of course, no fixed sum can be set for all localities, for it will vary with the location, but there is everywhere a limit of value. That limit should be the maximum charge for water right no matter what the cost of the works may be.

To appropriate money for irrigation works in excess of the probable return would be a radical departure for Congress to take, but it would be in line with other Federal endeavors, such as leveeing along the Mississippi, road appropriations, and harbor work. This should be the next step if it is intended to complete the reclamation of the West. There is no likelihood of private enterprise doing much more irrigation work, except in the better settled areas and along the Pacific Coast. This work is of sufficient importance to call for the help of the nation. There is no reason for restricting future work to the arid regions. Many swamp and overflow projects are of great importance.

In many places the burden of this cost above the sale value of water rights can be collected from the people of the immediate vicinity and not from the National Treasury. Reference is made to places where power development is possible; a small charge on such development would pay for storage and flood control and not be a burden on any one. Such conditions exist on the Colorado River, on the Columbia, and on other streams. A mill or two per kilowatt-hour would not add any considerable burden on industry or the population of the cities, but would furnish enough money to provide for reclamation works of great magnitude. It is the writer's opinion that the people of Los Angeles, Calif., and the surrounding territory would find it good business to pay 0.25 cent more per kilowatt-hour for power if that resulted in bringing into cultivation 1 000 000 or 2 000 000 acres of irrigated land in

Arizona and Southeastern California. In the Sacramento and San Joaquin Valleys irrigation districts are finding it possible to build storage works and sell the power to public utilities at a price large enough to pay the carrying costs on the reservoirs. Many reclamation projects have very little power of value. If, however, the power could be pooled it would go a long way toward carrying out reclamation enterprises, wherever they may be located, as fast as there is need for the land.

A rather close contact with both Federal Irrigation District and private irrigation enterprises, extending over a period of twenty-five years, has caused the writer to hope that the day is not far distant when Government operation of irrigation works will cease. Nearly five years of operating one of the first Reclamation Service projects and afterward the operation of a privately owned project for three years, and for the past ten years a close contact with a number of irrigation districts have led to the belief that the operation by the Government is the least desirable of all methods. The reason for this belief is not that there is any particular superiority in the methods or personnel, but on account of the attitude of the water users. Where the Government is operating there is no question but that a large and increasing percentage of the water users are hoping that some means will be devised whereby they can escape the payment of the debts which they have contracted to pay. The result is that the local politicians have an opportunity to work and something to work on. They, in turn, work on the larger politicians until enough noise is created to attract attention. The belief is prevalent that if this noise can be made loud enough and continued long enough, relief will be offered them no matter if there is no justice in the cause or any need of relief. The result is bad for the morals of the community. There is a tendency to complain, to ask for relief, for more facilities, to display little desire to get along with what has been provided until it is paid for, and almost no desire to pay off these debts to the Government.

The management of the projects is directed from Denver, Colo., and Washington, D. C., which is a bad thing, for long-distance management is never satisfactory. Rules and regulations for the governing of all projects alike have to be promulgated and in the course of time the project manager becomes little better than a chief clerk. The management should be turned over to the people as soon as responsible officers can be selected, and the Government rule from distant points should be conspicuous by its absence. To this end the irrigation district organization is being used with considerable success. One feels that, in the past, Reclamation Service operation has probably been more efficient, more fair-minded, and altogether better than could be expected from local managers, but the writer is thoroughly convinced of the final necessity of local management and assumption of responsibility by the water users.

Mr. Mead believes in the selection of settlers and the establishment of advisers to encourage and foster scientific farming and business-like selling of farm products. Selection of the settlers is certainly advisable. It is not altogether clear what formula has been devised to assure the selection of the

most competent men. Knowledge of agriculture, is not the only requisite. Energy, a large amount of "stick-to-it-iveness," and, most important of all, the mental quality which prevents discouragement in the face of failure or disappointment, are as essential as money. Records kept of the cause of success or failure of about 1200 settlers on irrigation projects led to the conclusion that lack of money was a small cause of failure. Plain discouragement, often before there was any reason for discouragement, caused 50% of the failures.

It may not be possible to select men with the requisite qualities of success. It has been tried, but not successfully so far. Certainly, selection can accomplish something toward raising the quality of settlers above that of those who have heretofore acquired homes on reclamation projects. There is, however, a great deal to be said about the equal rights of all citizens to share in the benefits of reclamation. A large percentage of men are incompetent and fail wherever they go and in whatever they undertake. Sometimes, it seems that it would be better to allow the incompetent to eliminate himself through the natural processes rather than to appoint a committee to do it for him by examination in a preliminary way. Either process is a painful one for the one who is eliminated, but when the cause of elimination is his own act, his appeals for consideration do not echo so loudly.

Many other factors than those which can be examined by the proposed committee for selection of settlers enter into the cause of success or failure of a farmer. There are in California two interesting land settlement colonies carried out by State funds on a plan credited to Mr. Mead. One of these colonies is a success, the other, a turmoil of agitation. Apparently many of the settlers on the latter must be eliminated before this colony can ever be made a success. No amount of capital would make successful farmers out of some of these men. There is a great difference in the physical conditions of these two colonies, but neither those differences nor the slump in prices are altogether responsible for the difference in the two colonies. Some of those subtle changes came about in the mental attitude of these people, and some of them have changed from hard-working and hopeful home-builders to hard-working appellants for further aid from the State. This sort of thing can happen anywhere, but it is most likely to happen when the State or Government is the party to whom the debt is due.

There is little tendency to avoid paying the kind of taxes that are collected by the county tax collector. If Reclamation debts are placed on the same plane as county taxes, a much larger percentage of collections will be made. It is questionable, therefore, whether or not selection of settlers will prevent failures. Failures occur in any business and it is probable that reclamation farmers are no more likely to fail than grocers or any one of many occupations. One must not expect the future of reclamation to be free from the frailties of human beings, nor is it fair to place the blame of present-day failures on the farmers of present reclamation projects. In the case of the California land settlement colonies a great deal of unfairness has been evident in the discussion of the situation, but this, being a State colony, came within the realm of State politics.

Following the selection of the settler, Mr. Mead urges the necessity of aid and direction in farm development and of co-operation in disposal of the products. At present, farmers receive such aid in nearly all parts of the country. There is greater need for such aid and organization in the West where markets are distant than in many other parts of the United States. The writer has always believed that such aid was necessary and has found it very helpful in many ways. In California, there are Farm Bureau organizations in nearly all the counties which are effective in spreading information about methods of farming and in forming marketing organizations. There is little need for any other organization although special problems are being effectively studied by the U. S. Department of Agriculture through its various bureaus, particularly that of Western Agricultural Extension. Every Reclamation project can well afford to support an experiment farm.

The field of reclamation does not seem bright when future development is considered. The difficulties which have centered around the Reclamation Service have been felt by every organization that has built Reclamation works. There is little encouragement to go ahead except in regions where there is a decided demand for land. The warmer sections and the coastal valleys close to large cities are able to go ahead without Government aid. In other parts of the country development is not going on. Many years must pass before there is sufficient demand for new farms to make it possible to proceed without direct subsidy.

In the last twenty years much has been learned about how reclamation should be carried on. It is necessary to select the land carefully so that none but the best be placed in the hands of farmers. It is necessary to select the farmers in order that the farms be placed in the hands of none but the most experienced men. It is then essential that these choice farms in the hands of selected men be organized and advised and directed until they become profitable, and until good habits, in an agricultural sense, become established. Even with these matters all attended to, there is no hope that the reclamation projects of the future will be free from failures. The number of failures, however, will be much reduced.

Direct subsidy of future reclamation when there is a demand for more land seems essential. Works cost more than the land is worth in many cases, at least more than the farmer can hope to repay in any reasonable time. The question of the future is whether work is to be stopped or whether Congress will provide funds in excess of the amount which can be collected from the water users. The extension of agriculture in the West is so important that Congress will be justified in direct appropriation.

Failures occur in any business and it is probable that reclamation projects are no more likely to fail than projects of any other many departments. One must not expect the future of reclamation to be free from the failures of human beings, nor is it fair to place the blame of present-day failures on the failures of present reclamation projects. In the case of the California land settlement colonies a great deal of unhappiness has been evident in the discussion of the situation, but this being a State colony, some within the realm of State politics, and not a Federal one, it is not within the

LAND SETTLEMENT OF IRRIGATION PROJECTS

Discussion*

By MESSRS. W. G. SWENDSEN AND CHARLES H. WEST.

W. G. SWENDSEN,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper is a valuable contribution to the meager information concerning this subject. Any discussion of it must be based, necessarily, on experience and observation since it is abstract, not to say complex, and cannot be made to yield to any concrete or scientific analysis. It would be surprising in a discussion of such a subject if wide differences of opinion did not develop, but, notwithstanding this condition, the writer finds himself largely in accord with the author, and will only endeavor, therefore, to elaborate on one or two matters of major importance.

It is no easy task to fabricate a farm from the desert. To accomplish the feat, a person must be equipped either with ample funds or with an unlimited supply of physical strength, endurance, courage, and with a mental development capable of directing these forces; after this is accomplished, the farm provided and improved, the larger problem—its successful operation and the enjoyment of life during the three hundred and sixty-five days each year that must be spent upon it—remains to be solved.

The matter of colonization, after all, is but mere traffic in human beings, and a full realization of the seriousness of the business must be appreciated if success is to be obtained. Success is used here in its broader sense and implies not only financial gain or prosperity, but happiness and contentment as well. To the extent that Government agencies, including States, cities, and municipalities, engage in colonization, it must be assumed that the purpose is to build good citizens, promote public welfare, happiness, and contentment, and add to the well-being of humanity and of the nation, generally. While other agencies are engaged in the business, and quite properly, for financial gain or business advantage, the real seriousness of the matter—traffic in human beings—should not be overlooked. Families should not be encouraged to abandon cities or other homes to take up life on the farm until it is definitely known that they are equipped and fitted for such a vocation. They should be equipped not only financially but, in addition, have general fitness for the new life they are to undertake. Education or understanding of one's work and sympathy or co-ordination with one's surroundings are essential to

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‡ Received by the Secretary, July 8, 1925.

both financial success and happiness. After all, these are the real essentials in farm life as in other businesses.

Primitive man, by reason of his limited mentality, was contented and happy with a full belly and a gratification of his physical passions. In the present highly intellectual age, understanding of one's surroundings and work are essential to success and happiness. There is no more interesting or wholesome work than that of farming under irrigation, dealing as it does with the natural elements of sunshine, soil and soil fertility, atmosphere, water, etc., and the bringing of these together in maturing plant life. The greatest return cannot be had from such a vocation unless the participant is equipped with, at least, a fundamental scientific knowledge of agriculture and the processes which the seeds and plants must undergo to reach maturity. It would, indeed, be a dull and uninteresting life to the engineer or the man in other professions if he were obliged to subscribe to mere formula in the building of structures and the doing of other tasks, and thus proceed without a definite knowledge of the conditions and things with which they necessarily deal. Likewise, it must be a dull life for the farmer, isolated as he is from extended contact with his fellow man, if he is obliged to devote himself to the mere physical task of getting results and is not equipped with the proper training to consider the interesting scientific end of crop production.

The irrigated farm, offering as it does conditions well adapted to intensive farming, the rotation of crops, and with it, the continued building up of soil fertility, and the adjustment of crops to meet market and other conditions, offers advantages not enjoyed in agricultural territories where irrigation is not practiced. With these advantages, however, the farmer also must assume added responsibility and, if success is had, must be equipped to farm under real scientific methods.

So long as the farmer must sit on his plow from day to day with no other thought than that of how nicely the furrow turns over, or how difficult his daily manual tasks are, farming will be a drudgery; but when he is able to understand and analyze the simple, but very interesting and beautiful processes through which plant and animal life pass to reach maturity, drudgery will cease and his daily tasks will become a real pleasure. When this condition is fully realized and satisfactorily solved, the greatest problem in colonization will, likewise, have found a satisfactory solution.

To cite just one glowing example: In 1910 a settler was observed building his house on a piece of land under an irrigation project in the West. The site was on sloping ground. He had not even taken the trouble to level up the foundation on which he was building, and was so inexperienced in matters of this kind that this necessity did not occur to him. After it was called to his attention, he readily yielded to the suggestion and with some assistance provided a level foundation. Later, in the building of the superstructure, this settler made the mistake of putting the weather-boarding on his house upside down. Similar experiences were had in the improvement of the farm and, finally, in its operation, until after a comparatively short time, his funds and courage were exhausted. He had wasted two or three years of his life and that of his family only to meet with dismal failure and to return to his

previous work in the city, that of operating a street car. This man was equipped with physical strength and mentality to perform the tasks on the farm, but neither of these energies were trained for such a pursuit. Had the two years wasted on the farm been spent in an agricultural college, or other institution, where the fundamentals of farm mechanics, agriculture, agronomy, soils, and other subjects, are taught, this farmer could have moved on a farm with even much smaller financial resources and probably would have succeeded.

In other words, if colonization on irrigated areas is to succeed settlers must be selected from persons who are trained and skilled in the art of irrigation and farming, and who are adapted by natural inclination and environment to the work which they are undertaking.

Assuming that the natural agencies exist, for example, that the soil, climate, market, and other conditions are favorable, that the water supply is sufficient, the irrigation works and systems are adequate and so permanent as to insure a continuity of service in the delivery of water, then the colonization, if done with the right kind of settlers, will succeed. Obviously, this is the only means by which the investment in any irrigation enterprise can be recovered.

It is rather unfortunate that, because of the selfish ambitions of promoters, citizens, communities, and other institutions, the tendency is to promote the reclamation and colonization of new or unoccupied areas in advance of the time when economic conditions, including a demand for increased farm production, really warrant such development. In view of present conditions, the policy should be to promote the interest and prosperity of persons already on the farm to the end that each farm may be made a good one and be put on a dividend-paying basis; if and when this is accomplished colonization of other areas can be had with little, or no, difficulty.

CHARLES H. WEST,* Assoc. M. Am. Soc. C. E.—With the exception of Southern California and the foothill fruit areas of the great interior valley, irrigation development in California is for the purpose of increasing the agricultural production of lands already farmed. Most of the small projects are now completed so that construction generally is for the extension of projects or the consolidation of several projects into one larger unit. The problems, therefore, are different from those of the projects of the arid region built for reclamation, for in California the proper type of agriculture is established and the best methods of irrigation practice are well understood. A change from dry farming to irrigated farming involves the subdivision of the land, the leveling, checking, and growing of general crops, a task less difficult than producing a farm out of virgin land.

The pioneer type of project settler is now nearly a thing of the past. A few years ago settlers could become established with little capital and could get along on a small cash income. They produced much of the food they consumed and the standard of living was simple. To-day, the family garden and home-cured meat are seldom used; the neighborhood grocery store and

* Acting Asst. Prof., Rural Institutions, Dept. of Agriculture of the Univ. of California, Oakland, Calif.

meat market provide the farmer's food; the standard of living is much higher; the family must have automobiles and a radio, and must go regularly to the "movies". Taxes are also higher due to the paved highways, the many modern school buildings, etc. Because of better transportation facilities and better marketing agencies, farm products are brought into competition with a much larger producing area. The keener competition with older producing areas and the increased standard of rural life demand better farming methods, more efficiency in farm operation, and better business methods.

It is almost impossible for a man with no capital to improve and pay for a farm. He must have money enough to improve the land when he buys it or capital enough to pay for his household and operating expenses while he is developing the farm. He cannot compete with others if only one-third to one-half his land is improved, except by greatly reducing his standard of living. To-day the farmer's family is not willing to remain on the land if too much hardship is experienced, for it is generally only a short distance to town where easier work can be obtained.

Owners of land for subdivision, bankers, and real estate men have continually refused to recognize the amount of capital needed to develop farm land. This is emphasized by the lack of interest shown in considering planned farm settlement. If raw land costs \$50 per acre and when improved acreage is worth \$200 per acre, where is the settler to get the \$150 per acre to put into the farm? The new settler usually has about 15% of the capital needed to purchase and improve his land, and, hence, his credit with the bank will be quite limited. Only through help and co-operation from the former owner can he hope to succeed. The former owner often rents (to the settler) farm equipment or additional land and by introducing him and speaking a good word for him to the local banker helps him to obtain credit, and often foregoes payments due him on the purchase of the land in order to help the new settler to become established. If the owner is not willing to co-operate to this extent he should at least make sure that the purchaser either has sufficient funds to see him through to complete development or is discouraged from the undertaking. To see the venture to half completion and there encounter privation and discouragement is unfortunate for both the former owner and the purchaser. Many land owners have found that failures on their projects are poor advertising and, therefore, expensive. It is being realized more and more that settlers who have a possibility of success must be selected and not merely those who have a strong desire to succeed. The principal requisite to success, aside from a strong ambition and an inclination to work hard, is the necessary amount of capital to develop the farm to full production, or the equivalent, an amount of credit at reasonable rates of interest to see the venture through.

To give a more definite idea of the capital involved in a typical farm development in California, the following illustration is presented. This example has been selected at random from the records of numerous farms that have been studied. This farmer purchased 34 acres of raw land in June, 1921, and now has 26 acres of the 34 developed. His investment in the farm consists briefly of the following:

34.23 acres of land at \$250 per acre.....	\$8 557.50
Pipe line distributing system.....	2 379.40
14½ acres of alfalfa, at \$33 per acre.....	478.50
4 acres of pasture, at \$25 per acre.....	100.00
4½ acres of vines, at \$235.05 per acre.....	1 057.71
2 acres of trees, at \$142 per acre.....	284.00
Buildings and barns.....	3 458.75
Farm equipment.....	739.60
Household goods.....	681.00
Livestock: 18 dairy cows, 5 heifers, 1 pure bred bull, and 1 team of horses.....	3 100.00
Total	\$20 836.46

The total expenditure represents \$585 per acre and a capital expenditure for improvements of \$335 per acre.

Because of refusal to recognize the large capital required to develop farm land no effort has been made to provide financial machinery to bridge the gap between the limited capital of the farmer and the first mortgage credit available when the land is fully developed. This problem merits earnest consideration.

Trouble also has been experienced from failure to appreciate the difficulties and the expense involved in improving raw land. The planning and developing of farm land can be done most economically by those experienced and trained in land development. The trained and equipped organization does the developing in a thorough way the first time, eliminates all waste of time in performing the necessary operations, and does away with the losses caused by doing over things that were imperfectly done the first time. Few farmers do their developing efficiently. The type of checking used may not be satisfactory, or the ditch system is inconvenient, or some of the leveling has to be done over. This is true with all farmers. It takes much more to get the farm layout into final form than was expected. This is only natural, as a farmer usually develops but one farm in a lifetime.

Compare the status of the ordinary tenant farmer, acquiring land, with that of the settler on the new project. The former usually rents until he knows the peculiarities of the farm and has saved sufficient money to make a down payment of 10 to 15% on the purchase price of the land. A contract of sale is obtained and for several years the operator applies his earnings on the purchase price. He works and saves his earnings from a plant the layout and capacity of which are familiar to him. Returns from farming are small. It takes a lifetime for a man to acquire and pay for a farm. A farm represents a large capital. It should take years to acquire such an estate.

The irrigation project settler with no more capital begins the purchase of a farm. The first year he has no income. His funds are exhausted when the farm is about one-third to one-half developed. He tries to develop the remainder of the farm and pay for the land with his plant working at one-third to one-half its capacity. No wonder hardships and misfortune overtake him and his family. In the past, lack of capital has been paid for by privation and

hardships. It has been an expensive sacrifice to all but those who get joy in subduing and conquering a difficult situation. Those who persevere and conquer have the pioneer spirit.

In the past six years of experience in studying irrigation projects and interviewing farmers to learn of their financial condition and to determine their financial requirements for complete development, the speaker has been deeply impressed with the enormous waste in land improvement and the gross miscalculation, by those improving land, of the capital needed for development. Possibly organizations promoting land settlement have refused to consider the capital needs of settlers because most settlers had less capital than their rough calculations showed to be required.

Besides making due allowances for the wastes and losses in developing the proper farm layout, in order to determine the capital requirements certain assumptions must be made of the income that may be expected from the land during the period of development. Paper computations of capital requirements can be made by those experienced in land development and colonization work, but to a large extent these figures are arbitrary and the assumptions far from accurate. Engineers make their estimates of the cost of construction based on yardage calculations, current costs of construction, and the usual allowances for contingencies based on past experience. Yet more of these estimates fall short of the costs than equal or exceed the cost of construction. It is highly desirable to determine from actual figures (1) the degree of success usually attained in ditching and leveling farm land; (2) the losses sustained in using temporary installations until capital is obtained with which to install permanent improvements; (3) what are the usual household expenses, incidental and contingent expenses; (4) what are reasonable yields; (5) how often should crop failures be expected; (6) of the crops produced for how much will the producer find a market; (7) what are safe prices to assume; and (8) how much outside employment can the settler find while improving his farm, etc. These data are essential to the project settler. It is easy to determine leveling, checking, and ditch costs for a large outfit of trained men using good equipment and having plenty of capital on which to operate, but the results for the farmer of limited experience in land development, working with poor equipment and with much less capital than he needs, is a far different consideration.

If the owner of farm land to be subdivided will plan and improve the farm units before sale he will eliminate the usual waste in land development, but in order for the farmer of limited capital to make good it will be necessary to sell him land on small initial payments, charge a low rate of interest, and extend the payments of principal over a long period of years. Not many land owners have the capital or the desire to undertake such a program. For owners of farm land who have not sufficient capital to improve and subdivide their property, and in reclamation projects where the land is owned by the Federal Government, some provision should be made for obtaining financial assistance for the settler who undertakes the development of farm lands. This financing agency must be associated with the management of the project and both must be kept free from politics. State universities, the Federal Reserve Board, and the Federal Farm Loan Board perform public service

free from politics. This agency must be kept as free from molestation from local bankers, politicians, and settlers' committees as possible, and must have authority to deal firmly with each settler as the merits of the case require. The management must be thoroughly business-like and each settler under contract should be compelled to follow out an approved program of farm development. The agreement must be flexible, but it must also be definite and binding and should not be changed except by authority from the financing agency and the project management. It is evident that assistance should be given only to those who have enough capital to put into the enterprise to assure their interest and close co-operation. Representatives of the management must check up the operations of the farmer to see that he lives up to his contract and performs in a thorough and proper manner the work he agreed to do. Competent agricultural advisers are a necessity for such development. Past experience has demonstrated that for successful operation of a project the settlers must have representation in the management.

There are no more misfits in agriculture than in any other industry, but due to a lack of consideration of the problems here mentioned, which are highly essential in planning land settlements, and also due to lack of appreciation of proper financial facilities to finance farm development, it has become a saying that "it takes two crops of settlers to make it possible for the third to stick".

IRRIGATION DEVELOPMENT THROUGH IRRIGATION DISTRICTS

Discussion*

BY MESSRS. RICHARD R. LYMAN AND J. B. LIPPINCOTT.

RICHARD R. LYMAN,† M. A. M. Soc. C. E.—This paper covers the subject so thoroughly that little concerning the character and importance of irrigation development through irrigation districts can be added.

The farmer in the humid regions is perhaps the most independent and, therefore, the most individualistic citizen in the United States. He plows and plants and harvests, independent of his neighbors. This immunity from the necessity of help from others has given these farmers an independence which under certain conditions may be a serious handicap.

Farmers produce life's necessities. They could control the food supply. Before a man will endure long the pangs of hunger, he will gladly give all he possesses for food. With this advantage if farmers were as shrewd and, shall it be said, as selfish, as many business men, they would "sit tight" and demand prices for foods that would make them the multi-millionaires—the great "captains of capital".

Will they do it? They will not. Why not? Because they cannot; they are too independent. They have not had to co-operate, they have had no experience working with others, they will not combine. In fact, their natures and training are such that they cannot work together—they cannot trust their fellow farmers. Where, however, irrigation is a necessity and the ditches to be built are greater than one farmer can construct, he is compelled to get the help of his neighbors.

Now that all the simpler irrigation systems in the West have been built by individuals, partnership enterprises, and private corporations, the day has come when public corporations must be called on to furnish for long periods the great capital needed. So stupendous are the dams, canals, power plants, and other parts of the irrigation systems remaining to be built that their construction demands not only the co-operation of the States, but that of the Federal Government.

Thus has been thrust on the farmers of the West the necessity of co-operation, the idea of working together, and the irrigation district is the latest and, thus far, when organized under favorable and proper economic

* This discussion (of the paper by E. Courtland Eaton and Frank Adams, Members, Am. Soc. C. E., presented at the Summer Meeting, Salt Lake City, Utah, July 8, 1925, and published in March, 1926, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Civ. and Cons. Engr. (Lyman & Pack), Salt Lake City, Utah.

conditions, the best co-operative institution for farmers that has been devised. It has in it the municipal idea. Practically the same stable machinery that controls and operates counties and cities also operates and controls these districts.

These organizations have many advantages: They operate under popular control; the organization can be formed without the distressing delays required when private contracts have to be made; no abstracts of title need be secured; no strong, dissenting minority can delay progress indefinitely; theoretically, at least, no profit is paid for promotion; the assessments for these districts are made the same as those for general taxes, and they are collected by the same agency. The obligations of the district have priority over private mortgage liens on the property in the district, and with the approval of the Secretary of the Interior the district secures control of even the public land that is located in it.

Such an organization necessarily has a standing in the financial world that individuals and private corporations cannot possess. Districts can co-operate readily with the U. S. Reclamation Service. They eliminate undesirable speculative features, and are able, because of their stability, to continue successfully when organizations without such standing might fail. Land operated under an irrigation district can be put under cultivation sooner, can meet assessments earlier, will have a more rapid development, and will, in a shorter time, reach the profitable stage.

While these are some of the advantages of an irrigation district, there are disadvantages. In a district definitely specified benefits are assessed against specified areas. Once made, this assessment, however unfortunate, however unjust, can be changed, it seems, only with very great difficulty.

In one district in Utah, so the speaker has been informed, the land owners organized, thinking that water allotments could be increased or decreased from time to time. It seems that many allotments were increased without the knowledge or consent of the land owners. Allotments were improperly and perhaps hurriedly made.

For example, the original cost per acre-foot of water is roughly \$100. The original area to be benefited was given as more than 7 000 acres. The actual area proves to be less than 4 000 acres. The bonds were sold at \$84. The required annual payment per acre is, in round numbers, \$16 for each acre-foot of water, and, therefore, for land having an allotment of 4 acre-ft., the annual cost per acre (and these payments will cover a period of twenty years) is \$64. It has been stated that 4 acre-ft. have been allotted to some land on which no water at all can be used. No one under these conditions would make such payments.

It looks as if all concerned in this district had in mind in the beginning that amounts allotted could be changed from time to time and that the final adjustment would be to the satisfaction of all interests. The Utah law, however, at once fixed these allotments so that the grossest injustice it seems cannot now be corrected. The law reads:*

* Session Laws, 1921, p. 191, Chapter 73, Sec. 11.

"The Board of Directors * * * shall * * * make final revision and allotment of available water * * * provided that * * * such final allotment may not thereafter be decreased as long as there may be any outstanding indebtedness in excess of 2% of the assessed valuation on the lands within the said district."

Then, again, if assessments in this particular district are not paid, the land, it is stated, cannot be sold because the metes and bounds of the areas on which the assessments have been made, have not been given in any case. The wording of the allotment is somewhat as follows: "Ten acres of a specific 40 acres has been allotted 3 acre-ft. of water." How can a deed for such a 10-acre tract be prepared?

Confusion, misunderstanding, and ill-feeling—these are the results thus far of the organization of this irrigation district. Tremendous financial losses also must come sooner or later to all who have a financial interest in it.

Certainly farmers are entitled to know the complete facts concerning a proposed irrigation district before they assume the financial obligations it imposes. The State or the Government, or the two together, should provide competent engineering service in all cases of this sort. The Mapleton and Springville Irrigation Districts, in Utah County, Utah, which are operating under the Strawberry Valley Reclamation Project, seem to be pronounced successes.

The irrigation district is a boon to irrigators. Experience will soon disclose the imperfections in the present law and present methods of organization. With these corrections, irrigation districts will be powerful agencies for promoting co-operation among irrigators and for advancing agricultural interests generally.

J. B. LIPPINCOTT,* M. Am. Soc. C. E.—The State of California now has had thirty-eight years' experience in organizing and operating irrigation districts. Of this the first ten or fifteen years were unsatisfactory because the law was made the medium of speculation in land and water. For the last twenty-two or twenty-three years there have been a series of amendments to that Irrigation Act that are particularly directed to the enforcement of the collection of the assessments and obligations of the irrigators. Probably the most effective factor has been the appointment of a State Bond Certification Board composed of the State Engineer, the Attorney General, and the Superintendent of Banks of the State, who, with their respective expert knowledge and abilities, are able to pass on the legal status, the engineering features, and the financial features of the irrigation districts for presentation to State authorities.

It has been stated that 2 000 000 acres have been irrigated by the Federal Reclamation projects. California has irrigated more than twice that acreage under the State Irrigation District laws. There are now 98 irrigation districts in California. The breaking up of large land holdings has come automatically so that, in 52 districts of which the speaker has records, the average area of a farm is 58 acres, which is a small farm unit; but the small farm unit is the ideal one and the one which the Federal projects are endeavoring to approach or reach.

* Cons. Hydr. Engr., Los Angeles, Calif.

The enforcement of the collection of the assessments for interest and operating expenses has been made most effective. The primary authority is the Board of Directors of the District, but if they fail in carrying out their duty, the County Board of Supervisors and the County Attorney are required under their bond to carry out and enforce the necessary acts for the collection of these moneys. If the County authorities fail, then the Attorney General of the State is, under the law, obliged to proceed in the collection of the necessary funds for the payment of these charges. If the taxes are not paid, the lands are sold by these authorities. As a result the irrigation district bond in California to-day is considered a more secure investment than the ordinary municipal or school district bond.

In California only 1.47% of the area of the irrigation districts—of more than 4 000 000 acres—is in any way delinquent in the payment of their obligations; but 1.94% of the bond issues are temporarily delinquent. According to the bondholders and the State authorities these small adjustments are well under way toward payment.

The irrigation district bonds in California are on a $5\frac{1}{2}$ to $6\frac{1}{2}$ % basis whenever they are approved by the State Bond Commission. If they are not approved by the Commission, they are not marketable. As far as irrigation problems are concerned, the State Irrigation District is a satisfactory and effective institution. It has its defects just as other large institutions have, but it is a success. All this refers now to "The California Irrigation District Act". There are other broader Acts that provide for co-operation between the various districts, political units of the State or subdivisions, or with the Federal Government if desired.

This record is of especial interest in connection with the various reports of the failure to make adequate and prompt collections on these Federal and Reclamation projects.

In the selection of these Federal projects, the engineer was there, of course, with his reports and advice; but the Congressional delegations—the delegations of Chambers of Commerce, delegations of farmers, delegations of citizens who went to Washington and pressed their claims on Congress and the Secretary of the Interior—they were largely the controlling factors in obtaining the decision of the Secretary of the Interior, which is the final decision in the "adoption" of the project. The engineers were all "good fellows" as long as they had money to spend, but when the money was gone, these same delegations were back in Washington with all their reasons why the bills should not be paid and why these contracts should not be carried out on the part of the irrigators—reasons why they should not pay for these projects that they have had built, without interest. It was that pressure of "statesmen" and of politicians that was instrumental in encouraging this delinquency. The engineer has been a very convenient "goat".

While this "Fault-Finding" Commission has gone throughout the country giving a volume of its reasons why these payments have not been made, the real reason is because they have not been insisted upon. California has the same sort of farmers with much the same crops, but they have paid their bill because they have had to pay it. If the same policy were adopted through-

out arid America by the Reclamation Service it would have had the same experience as California. The engineer is not responsible for this situation.

The Worst conceivable way to go before Congress or State authorities to obtain additional money for the expansion of irrigation enterprises in arid regions is with a vast number of repudiated contracts. Before getting more money for big enterprises, like the Colorado River or the Columbia River projects, the thing to do is to pay the bills for the work that has already been done.

... funds for the payment of these charges. If the taxes are not paid, the bonds are sold by these authorities. Again, the irrigation districts bond in California today is considered a more secure investment than the ordinary municipal or school district bond. In the same way, the bonds of the California only 1-45% of the even of the irrigation districts and more than 1,000,000 acres—this was the amount in the payment of their obligations; but 1-25% of the bond issues are temporary obligations. A condition to the bonds is that the State authorities should not repudiate any of the bonds. The irrigation districts in California are not in a position to repudiate any of the bonds approved by the State Bond Commission. If they are not repudiated by the Commission, they are not repudiable. As far as irrigation problems are concerned, the State Irrigation District is a satisfactory and effective institution. It has its defects just as other large institutions have, but it is a success. All this is now to be done by the California Irrigation District Act. There are other agencies that provide for cooperation between the various districts, political units of the State or subdivisions or with the Federal Government if desired.

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While the "Last Frontier" Commission has come throughout the country during a volume of its reports, the reasons why these payments have not been made, the reason is because they have not been insisted upon. California has the same sort of farmers with much the same crops, but they have paid their bills because they have had to pay. If the same policy were adopted through-

PROGRESS REPORT OF SPECIAL COMMITTEE ON FLOOD-PROTECTION DATA

Discussion*

BY MESSRS. MORRIS KNOWLES AND ROBERT E. HORTON.

MORRIS KNOWLES,† M. AM. SOC. C. E.—The Committee is to be commended for going on, undiscouraged in the face of difficulties. It is trying to accomplish a very worthy piece of work. All who have had to do with the study of flood conditions know how difficult it is to interest people sufficiently as to the seriousness of the situation.

As the Committee has stated, the theory of probabilities seems to fit, and this is one of those things on which people like to base their guesses, even if there are mathematical principles which apply; and people will continually guess that they are not going to be affected for some time. The City of Pittsburgh, Pa., is suffering from such a delusion at present, because, while considerable progress has been made, those who are familiar with the situation have an acute realization that, in order to arouse people, a startling catastrophe is necessary.

Even without such a startling catastrophe, an authorization for a survey has been obtained, thanks to Governmental and State appropriations. Through the co-ordination of those interested in floods and those interested in city planning, a report has been submitted which will bring to the attention of the citizenry the necessity for a large sum of money to be included in a bond issue for protective works. Incidentally, much has been accomplished in the so-called prevention of floods by the building of many reservoirs on the watersheds, under the auspices of private capital. Private capital is interested in the erection of reservoirs for the development of water power; and those reservoirs have contributed not a little to the diminution of destructive floods.

Any one who can do anything for the promotion of accurate studies on flood matters anywhere should do so. It is an object that is worthy of the support of all members of the Society.

ROBERT E. HORTON,‡ M. AM. SOC. C. E.—The collection of flood data is a proper Federal function and the Federal Government should be persuaded to provide funds to do the necessary work. There is a large volume of flood data in existence which has never been completely compiled and analyzed. It

* This discussion (of the Progress Report of the Special Committee on Flood-Protection Data, presented at the Annual Meeting, January 20, 1926, and published in March, 1926, *Proceedings*), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

‡ Cons. Hydr. Engr., Albany, N. Y.

seems proper to make the suggestion that co-operation be secured with power companies with reference to furnishing flood data. Many power companies record floods and flood conditions on streams where they operate. If there is a central bureau organized and maintained for the purpose of collecting and publishing these data, the power companies will usually be more careful to collect the data than if there is no outside interest.

One advantage of obtaining records from power companies is that they always have men on the ground, available to take the necessary readings, and their self-interest compels them to watch maximum floods closely. Another advantage is that at present most of these companies have dams of excellent construction and usually with ogee cross-sections, for which reliable discharge coefficients are now available. There need be no serious error in the selection of a discharge coefficient for a well-constructed ogee dam, especially for high heads and flood conditions. In fact, coefficients of discharge over dams are more reliable for high heads than for slight depths of overflow. On the other hand, the current meter is not well adapted to securing maximum flood discharges. For example, in the procedure of the U. S. Geological Survey, one of its field engineers will have possibly twenty to one hundred locations in his territory where he would like to obtain the peak or crest of some particular maximum flood. If he has half a dozen assistants and current meters he will fall far short at best of obtaining all the available data.

The use of a current meter in a flood is, however, subject to difficulties, especially in the Northern States. If a great flood occurs at a time when ice is going out the current meter may be practically useless. At other seasons of the year the flood may carry drift and débris to such an extent as to make it impracticable to use a current meter, except perhaps to measure the surface velocity of the stream. Even if the stream is clear of ice and débris, it usually requires more equipment and assistants to obtain good results in measuring a maximum flood discharge at a given gauging station than for measuring ordinary and low discharges. This results from the tendency for the meter to swing down stream with the higher velocities during floods, the likelihood of there being silt in the water, and the fact that during floods the cross-sectional area is much larger than at ordinary stages and there are likely to be side cuts and overflow channels often more or less obstructed with brush.

In the application of data of maximum flood discharges there are two things which an engineer may want to know: First, as to the frequency of floods of different magnitudes on a given stream; and, second, as to the absolute maximum discharge which Nature can produce from a given drainage basin.

The writer is firmly of the opinion that there is a maximum or limiting flood discharge for each location on any given drainage basin. In other words, Nature can no more produce a Mississippi River flood on the Hudson River than an ordinary barnyard fowl can lay an egg a yard in diameter, and for very much the same reason—it would transcend Nature's capabilities under the circumstances.

Probability methods as ordinarily applied do not always lead to conclusive results in this respect. There are probability curves and probability curves. Many types of probability curves indicate an infinite magnitude of the event if the record is only continued for a sufficiently long period. Other types approach an asymptotic or limiting value of the magnitude of the event. In the writer's opinion only the latter are properly applicable to phenomena dependent on rainfall, such, for example, as flood discharges. There are several probability formulas of this type. One which has been developed by the writer and which seems to give excellent results with reference to flood discharges takes the form:

$$R = a(1 - e^{-kt^n})$$

in which, a , k , and n are constants for the particular stream and location; t is the average exceedance interval, in years, for a flood having the magnitude, R ; and R is the limiting flood discharge expressed as a ratio with respect to the average annual flood on the same stream.

Examples of the application of this frequency or probability formula to annual rainfall and flood records show that it gives results in good agreement with observed series of data. To use this formula, merely a reasonably long record is required from which to determine the constants. In other words, it is possible from a flood record of moderate length to determine by this means approximately the true maximum or limiting flood discharge for the given stream and location*.

* For examples of application to rainfall records, see the paper entitled "Determining the Mean Precipitation on a Drainage Basin," by Robert E. Horton, *Journal, New England Water Works Assoc.*, Vol. 38, March, 1924.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WALTER MASON CAMP, M. Am. Soc. C. E.*

DIED AUGUST 3, 1925

Walter Mason Camp, the son of Treat Bosworth and Hannah A. (Brown) Camp, was born at Camptown, Pa., on April 21, 1867. He came of Colonial ancestry; his earliest paternal American ancestor was Nicholas Camp, who came from Essex County, England, in 1631, with the Rev. John Elliott, and settled in Massachusetts. Both his paternal and maternal ancestors served in the Revolutionary War. His father who was an insurance surveyor and the author of insurance literature, served as Captain of Company F, 52d Pennsylvania Infantry, during the Civil War and was once confined in Libby Prison.

Mr. Camp received his preliminary education by winter attendance at the Public School in Wyalusing, Pa. At the age of nine he was employed as a fireman in a planing mill at Wyalusing and, later, for four years, he worked on farms and at lumber camps. From 1882 to 1887, he was with the Lehigh Valley Railroad Company as Trackman, Chainman, and Rodman on preliminary and location surveys, and double-track construction, and, later, on extensive river surveys for bridge location. During this time he acquired also a working knowledge of telegraphy.

In 1887, Mr. Camp entered Pennsylvania State College, from which he was graduated in 1891 as Civil Engineer. For a few months following his graduation he served as a Surveyor with the San Joaquin Valley Railway Company, at Fresno, Calif., and as Draftsman in the office of the Chief Engineer of the Southern Pacific Company, at San Francisco, Calif. From 1892 to 1894, he was Engineer and Superintendent of the Rainier Avenue Electric Railway, Seattle, Wash., which extended eight miles into the suburbs; he had full charge also of the location and construction of an uncompleted part of the road. He designed and built rolling stock for heavy freight traffic, repair shops, station buildings, and wharves for the steamers operated in connection with the railway and also designed and rebuilt a counterweight system for operating electric cars on a 17% grade.

Mr. Camp was with the Seattle, Lake Shore, and Eastern Railway Company as Engineer in charge of building spur-tracks into lumber camps in 1894 and 1895. His work here consisted in relocating and rebuilding parts of the main line, widening cuts and embankments, and reballasting parts of the roadbed. He served also as Worktrain Foreman, Surveyor, and Section Foreman. In 1895, in addition to other work, Mr. Camp studied as a post-graduate student in electrical and steam engineering at the University of Wis-

* Memoir compiled from information on file at the Headquarters of the Society.

consin, and, in 1896, taught for a short time in the National School of Electricity in Chicago, Ill. He then accepted the position of Receiver and Inspector of Materials for construction, and served as Superintendent of Construction, for the Englewood and Chicago Electric Railway Company (a storage battery road), in Chicago. He also had charge of building the counterweight system for this Company in Morgan Park, Ill.

In the spring of 1897, Mr. Camp became Engineering Editor and, later, Managing Editor of the *Railway and Engineering Review* (now the *Railway Review*), in Chicago. He served also as Consulting Engineer to the Englewood and Chicago Electric Railway Company, with reference to the operation of the counterweight system and in repairing it.

He acted as Consulting Engineer to capitalists investigating the cost of construction, operation, and traffic possibilities of an electric railway for Towanda, Pa., in 1898, and, in 1899, to the Automatic Rail Joint Spring Company of Chicago in reference to track fastenings and the manufacture of concrete ties. In 1900, he was consulted by the Koku Railway Company of Japan in the matter of the selection of track materials.

Mr. Camp was admired as a writer by members of the railroad fraternity because of the clear and forceful presentation of his opinions. He was particularly well versed in matters relating to track maintenance and automatic train control. On learning of his death, one of his associates remarked: "He was the best-informed man I ever knew". His published works, excluding his editorials, comprise "Notes on Track," which is used as a textbook in colleges having Railroad Departments; "Life of Samuel F. Patterson"; and "Railroad Transportation at the Universal Exposition, St. Louis 1904". He was also the author of numerous papers published by engineering and historical associations. No one was ever in doubt as to the exact meaning of his editorials, which were based on thorough investigation and sound knowledge. He understood railroad conditions thoroughly.

Although devoted to the work of his profession, Mr. Camp was a man of varied interests. He was fond of outdoor life and owned and operated a farm at Lake Village, Ind., where he became a successful farmer. He also had a quiet retreat in the Michigan woods. He was a student of Indian life, customs, and history. He spoke a number of Indian languages and dialects and had made a study of their wars with the white man from the standpoint of the Indian. His vacations were often spent in research work on their reservations in the West where he was a welcome visitor, and it was not unusual to see a group of Indians in his office seeking advice. Mr. Camp was also a student of Astronomy and was interested in the question of whether or not the planets are inhabited. In his will he provided a fund which is to be used by the Northwestern University to carry on research work to investigate this problem.

He was greatly concerned in the welfare of young men, many of whom profited by his kindly counsel. His cabin in Michigan and his farm were always at their disposal. He was married, at Blue Island, Ill. on May 2, 1898, to Emeline L. F. Sayles, who survives him.

He was a member of the American Railway Engineering Association; American Railway Bridge and Building Association; Roadmasters' and Maintenance-of-Way Association; Society for the Promotion of Engineering Education; Permanent Way Institution (Great Britain); Railway Signal Association (now Signal Section of the American Railway Association); Mississippi Valley Historical Association; Western Society of Engineers; Chicago Engineers Club, and others.

Mr. Camp was elected a Member of the American Society of Civil Engineers May 1, 1901.

MARTIN RYERSON EVERETT, M. Am. Soc. C. E.*

DIED FEBRUARY 4, 1926.

Martin Ryerson Everett was born at Frankford Township, near Branchville, N. J., on August 21, 1867. He was the son of Martin Ryerson and Julia (Roe) Everett, both of whom died before he attained the age of ten. He attended the public schools of Branchville and, after graduating, entered the employ of one of the merchants of that town.

In 1890, becoming dissatisfied with the limited opportunities of a rural community, Mr. Everett went to New York, N. Y., and accepted a clerical position with the Jackson Architectural Iron Works. While thus employed he met many civil and mechanical engineers and after learning of their achievements became so interested that he decided to become an engineer. He immediately entered the evening sessions at Cooper Union and received the degree of Bachelor of Science in 1898. At this time he was in charge of the Structural Shop of the Jackson Architectural Iron Works.

In 1900, Mr. Everett went to Newark, N. J., to be Shop Manager and Designer of Steel Structures for Cooper and Wigand, Contractors and Engineers. After receiving the degree of Civil Engineer from Cooper Union on May 28, 1904, he became connected with the Cooper-Wigand-Cooke Company.

From 1906 to 1908, Mr. Everett was General Manager of the Cooper Iron Works, of Newark. Resigning from this position he engaged in private practice as a Contractor and Engineer, continuing as such until 1910, when he became Vice-President of the Hedden Iron Construction Company, in charge of operation. After serving as Vice-President with this Company for two years he resigned and again engaged in private practice.

In 1914, he incorporated the Company known as the Martin R. Everett, Incorporated, Engineers and Contractors, with headquarters in Newark, and served as President until his death from a lingering illness on February 4, 1926.

Mr. Everett had a wonderful and unique personality, made many friends, and was a favorite with those whose good fortune it was to know him well. Always interested in sports and athletics as a young man, he was an enthusiastic supporter of them throughout his life and was never happier than in witnessing some big game.

* Memoir prepared by Victor Graff, Pres., Martin R. Everett, Inc., Newark, N. J.

Mr. Everett had resided in East Orange, N. J., for twenty-five years. He was an active member of the Arlington Avenue Presbyterian Church of that city, and was also a member of many fraternal and community organizations.

He is survived by his wife, Lillian Robertson Everett, his son, Martin Ryerson, Jr., and his sister, Miss Jennie H. Everett, of Bayonne, N. J.

Mr. Everett was elected a Member of the American Society of Civil Engineers on April 14, 1919.

FRANKLIN IDE FULLER, M. Am. Soc. C. E.*

DIED DECEMBER 16, 1925

Franklin Ide Fuller, the son of Leonard F. and Mary I. Fuller, was born on May 29, 1858, in Providence, R. I., of an old New England family. He attended the public schools, afterward becoming a student of civil engineering while working in the offices of the City Engineer of Providence. He spent four years in this office, winning several promotions, and mastering much of the technique of his chosen profession. At the end of his term, he entered the railroad service and was engaged in location and construction work on the New York, West Shore and Buffalo Railroad.

In 1883, Mr. Fuller went to Oregon and there was connected with railway work as representative of the Northern Pacific Terminal Company. The failure of the Northern Pacific Railway Company and the cessation of railroad construction in the Northwest at that time forced him to enter the contracting business in which he continued for four years, devoted largely to railway and heavy timber work. The next three years were spent with the Oregon Iron and Steel Company at Oswego, Ore., first, as Assistant to the Manager and, later, as Manager of the Company. Following this engagement he devoted a year to the real estate business.

In 1892, he became connected with the Portland Cable Railway Company, and remained with this Company and its successors, through several consolidations and mergers, until his death. He was active in the transportation development of the city, having done more perhaps to increase street-car lines in Portland than any other man. When the Cable Company became the Portland Traction Company, Mr. Fuller was made Manager and continued in this office until the Company was merged with the Portland Railway Company, when he was made General Manager.

By his forethought and hard work he saved his Company from failure during the panic of 1893. Banks, stores, hotels, and all lines of business were being closed. It was the blackest hour of Portland's history. Mr. Fuller at once put the line on as low an operating basis as possible. In order to get sufficient cash to keep it going, he sold books of street-car tickets among the business houses, even taking them into the residence districts and selling them from house to house. In 1904, came the amalgamation of the Portland Railway Company and the City and Suburban Railway Company, forming the

* Memoir prepared by J. P. Newell and D. C. Henny, Members, Am. Soc. C. E.

Portland Consolidated Railway Company. Mr. Fuller was General Manager for a year, then the properties were purchased by the Clark and Seligman interests of Philadelphia, Pa., and New York, N. Y., and the Portland Railway Company was re-organized with Mr. Fuller as President.

The incorporation of the Portland Railway, Light, and Power Company came in 1906, with the addition of the lines and other property of the Oregon Water, Power, and Railway Company. Mr. Fuller became Vice-President in charge of the railways of the Corporation.

One of the monuments to his engineering ability and progressiveness is the Portland Heights Street Car Line. At the time it was projected, it was said to be impossible, and impracticable, but his professional eye saw the way clear to the fulfillment of his plan, and for years this was the most scenic city street-car line in America.

At the time of his death, Mr. Fuller was the oldest official in electric transportation work on the Pacific Coast, both in length of service and in age. He was active in the affairs of the Chamber of Commerce, at times serving as Director. He was also a member of the Woodmen of the World, Modern Woodmen of America, Royal Arcanum, Arlington Club, and Multnomah Club. He was a Director of the Lumbermen's Trust Company and of the Oregon Portland Cement Company.

On April 14, 1886, he was married to Anna Jessie Parrish, of Portland, who survives him, with one son, Leonard F. Fuller, Chief Engineer of the Thompson Radio Corporation, of Ridgewood, N. J.

The outstanding feature of Mr. Fuller's professional career was his work as an executive, with the task of administering the intricate affairs of a great public utility. In this position the many-sidedness of the man was apparent. In the settlement of the innumerable controversies which arise between city officials and a street railway company, Mr. Fuller's word was always conclusive. Whatever he promised he performed.

The esteem in which he was held by the City Council of Portland was expressed in the following resolution:

"On account of his long service in this community and his intimate knowledge of local transportation and traffic problems, the counsel of Mr. F. I. Fuller was always valuable in assisting us to arrive at a fair and reasonable solution. He was a man of exceptional ability, sound judgment, and high moral character, always patient and considerate of the rights of others. Even in the face of strenuous objections he was always calm, deliberate, and courteous in presenting his views, or in pressing the claims and wishes of the large utility he represented. We voice our deep regret at his passing, because we realize that Portland has lost a valuable citizen, and the profession of engineering an honorable and worthy member."

Modern conditions of employment offer many opportunities for friction with subordinates, but the character of Mr. Fuller's relations with his men is best evidenced by their spontaneous tribute in a memorial addressed to Mrs. Fuller, in which they stated, in part:

"In the passing of Franklin Ide Fuller, we, his co-workers in the Portland Electric Power Company, feel keenly the loss of a faithful, sympathetic friend who was ever ready to advise and help us.

"His high sense of honor and loyalty to us and to the Company he so generously served have been an inspiration to all with whom he came in contact. We know no act of his life that does not express kindness, tenderness, and generosity.

"Any one who could occupy, with such gentle humility, the position he has so long and honorably filled is an example of the highest type of Christian character and is indeed a great man."

The esteem in which Mr. Fuller was held by his associates was voiced by the President of his Company as follows:

"He was well known and admired for his staunch qualities and rugged character by many thousands of citizens in all walks of life. He was quick to realize the needs of Portland and he always endeavored to serve his city and his fellow-men to the best of his ability.

"During the many years in which I have been associated with Mr. Fuller, his calm serenity, sure poise, and good judgment in the face of perplexing problems have always been a source of deep inspiration to his associates in our company. I have never known a more faithful friend, a more lovable companion or a more conscientious, loyal business associate."

The Directors of the Lumbermen's Trust Company expressed their sorrow in a memorial which is, in part, as follows:

"In the passing of Mr. Franklin Ide Fuller, the Lumbermen's Trust Company and its affiliated corporations have suffered the loss of a valued adviser, who, by reason of his broad acquaintanceship, his acute analysis of problems, his matured judgment and his unwavering integrity, has contributed largely to the material progress of our institutions. Beyond and above this his unselfishness, cheerfulness, modesty of character and boundless faith in his fellow-men have endeared him to each one of us and have left to us a loving memory which we will always retain."

Mr. Fuller's whole life was permeated by principles to which his Pastor and intimate friend, Dr. H. L. Bowman, bears testimony:

"Mr. Franklin I. Fuller during his residence in Portland was associated with the Church and with Christian work. In 1909 his membership was placed in the First Presbyterian Church with which he was actively affiliated. For the last thirteen years of his life he was a member of the Board of Trustees of the Church and for the last two years its President. He gave generously and loyally of his time and interest to the work of the Church.

"But Mr. Fuller's life was not one in which Christian character simply showed in Church activity. His life was an outstanding instance of the principles of the Master interpreted in terms of practical business life. In the beautiful and considerate fellowship of the home, in devoted friendship, in the sterling integrity of business conduct, in unselfish consideration of co-workers, in the activities of citizenship Mr. Fuller revealed the rich fruitage of Christian character. His was a life in which religion was woven so thoroughly as to become instinctive; with no impulse for display his life was yet a vocal testimony of the value of Christian ideals in building character and enriching personality.

"Such a life so mastered by the spirit of Jesus Christ becomes in memory as it was in fellowship a summons to the noblest life."

To the members of his profession Mr. Fuller's conspicuous success as an executive was a source of pride, his cordiality and kindness a constant en-

couragement, his fine public spirit an example, and his upright life an inspiration.

Mr. Fuller was elected a Member of the American Society of Civil Engineers on January 6, 1886. He was one of the first Presidents of the Portland Section of the Society.

JOB ROCKFIELD FURMAN, M. Am. Soc. C. E.*

DIED SEPTEMBER 7, 1925

Job Rockfield Furman, the son of John M. and Virginia Holmes Furman, was born in Westchester, N. Y., on June 21, 1865. His early schooling was received at the Friends School in New York, N. Y. He then entered the Stevens Institute of Technology at Hoboken, N. J., from which he was graduated in 1885 with the degree of Mechanical Engineer.

After a brief experience in the rebuilding of marine engines, in 1887, he entered the employ of Otis Brothers and Company, manufacturers of elevators and hoisting machinery. Working first as a Draftsman, he early acquired a considerable knowledge of elevator apparatus and was generally assigned to take charge of elevators of an unusual character.

In 1888, he went to Paris, France, as Resident Engineer in charge of all work in foreign countries. There he was in charge of the erection of the two elevators in the Eiffel Tower. This undertaking, at a time when prevailing low, brick-walled buildings did not call for any long lifts, presented a unique problem, since the Eiffel Tower elevators ascend 420 ft., with a load equal to 50 persons.

Mr. Furman remained in Paris during the time of the Exposition in 1889 and then returned to the United States where he took an important part in the development of high-pressure hydraulic apparatus, which was in vogue at that time. As Assistant Engineer, he co-operated in the design of three 25 000-lb. passenger elevators, as well as towers and viaducts, for the Weehawken Viaduct at Weehawken, N. J., and superintended its erection. About this time he was also engaged on the Catskill Mountain Incline. In 1891, he assumed full charge of the Engineering Department of Otis Brothers and Company with headquarters in New York.

In 1894, Mr. Furman again crossed the ocean, this time to Glasgow, Scotland, where, as Resident Engineer, he superintended the installation of twelve high-pressure 15 000-lb., vehicle elevators for the Glasgow Harbor Tunnel, the most powerful lifts attempted to that date.

In 1896 he severed his connection with Otis Brothers and Company to become Secretary and Treasurer of the C. F. Parker and Company until 1898. During this time, in addition to other works, he was in charge of the deepening of 6½ miles of the Erie Canal. He then went to London, England, where he engaged in professional engineering work as a Consulting and Contracting Engineer. As Consulting Engineer to the Sprague Electric Elevator Com-

* Memoir prepared by Theodore V. Purcell, Vice-Pres., The Peoples Gas Light & Coke Co., Chicago, Ill.

pany, he designed forty-eight 17 000-lb. electric passenger elevators for the \$15 000 000 London Central Railway and supervised their installation.

In 1904, Mr. Furman again became associated with the Otis Brothers Elevator Company as Assistant Chief Engineer and was placed in charge of the installation of the 170 electric lifts in the new London Underground Railways. He laid out the twelve large elevators installed in the Elbe Tunnel at Hamburg, Germany, and shortly afterward left the Otis Company to join the well-known architects, D. H. Burnham and Company.

With this firm, which later assumed its present name of Graham, Anderson, Probst, and White, Mr. Furman remained until his death. As Chief Mechanical Engineer he assisted in the design of many large buildings for which the firm acted as Architect, among which were the Peoples Gas Building, Wrigley Building, Continental and Commercial National Bank Building, Straus Building, the Federal Reserve Bank Buildings, the Field Museum of Natural History, and the Chicago Union Station, in Chicago, Ill., which was the last large work with which he was associated, and the Equitable Building, in New York, N. Y.

Mr. Furman died at the Presbyterian Hospital in Chicago on September 7, 1925, following a brief illness. His wife, Florence O'Mullen Furman, whom he married in London, had died in 1908, leaving no children. He is survived by three sisters and two brothers.

Although Mr. Furman's professional life was one of distinction and outstanding success, his private life was perhaps even a greater achievement. He was a man of strict integrity and of a notably affable and kindly nature. His constant cheerfulness and good humor, and this despite a chronic illness in his later years, won for him a host of intimate and devoted friends.

He was a member of the University Club of Chicago and the Engineers Club of New York.

Mr. Furman was elected a Junior of the American Society of Civil Engineers on July 2, 1890, and a Member on December 7, 1904.

CHARLES WILLIAM KNIGHT, M. Am. Soc. C. E.*

DIED MAY 20, 1923

Charles William Knight, the son of Joseph H. and Julia E. (Butts) Knight, was born at Stanwix, near Rome, N. Y., on October 26, 1847. He was graduated from the Rome Academy, having had among his teachers the Hon. Elihu Root and Professor Oren Root. He also attended the Eastman Business College.

Mr. Knight's father was a land surveyor and carpenter and under his instruction the son learned the elements of surveying and gained a thorough knowledge of the carpentry trade, which stood him in good stead in his later engineering work. He familiarized himself with the cabinet maker's trade also, in which he was able to do highly creditable work. His next earliest

* Memoir prepared by Charles C. Hopkins, M. Am. Soc. C. E.

experience in civil engineering was under the late George W. Chase, City Engineer and Surveyor of Rome, N. Y. While with Mr. Chase, he determined to make civil engineering his life work and applied himself to such studies as would fit him for that profession. He was associated with the late Peter Hogan, a Civil Engineer of Albany, N. Y., on several projects, among which were the water-works systems of Grand Rapids and Muskegon, Mich., and the original water-works system of Gloversville, N. Y.

After the completion of these works, Mr. Knight, as Contractor, built considerable parts of the sewerage system of Gloversville, and, later, became associated with the late Adam Miller, of Saratoga Springs, N. Y., and built under contract many systems of water-works. In 1881-82 he served as Engineer for R. D. Wood and Company, of Philadelphia, Pa., Contractors, on 6 miles of 48-in. pipe line from the Kensico Dam of the Bronx River supply. The partnership with Mr. Miller continued until about 1885, after which Mr. Knight discontinued contracting to devote his time to the practice of his chosen profession, making a specialty of Hydraulic and Sanitary Engineering.

A partnership was formed with Charles C. Hopkins, M. Am. Soc. C. E., in December, 1887, which continued until January 1, 1890, when J. W. Ledoux and J. W. Kittrell, Members, Am. Soc. C. E., were added to the partnership, forming The Stanwix Engineering Company, of Rome, N. Y. Mr. Ledoux retired in the fall of 1890, and Mr. Kittrell in 1896, the firm then becoming Knight and Hopkins. This latter association continued until 1910. In 1912, with his son, Arthur P. Knight, Mr. Knight organized the engineering firm of C. W. Knight and Son, which continued until the death of the elder Mr. Knight.

During his long career, Mr. Knight served as Chief Engineer of many complete systems of water-works, among which were those of Sayre and Athens, Scottdale, Uniontown, Muncy, Marietta, Tunkhannock, Connellsville, Greenville, and Uniontown, in Pennsylvania; Cambridge, Cazenovia, Frankfort, Ilion, Jordan, Ossining, Skaneateles, Wellsville, and Cleveland, in New York; and Newcomerstown, in Ohio. He was also Chief Engineer of the storage and distribution reservoirs of the Altoona, Pa., Water-Works; of the Westmoreland Water Company's System supplying Greensburg and other Pennsylvania boroughs; of the Bear Rock Run and the Bell's Gap Run Reservoirs for the Pennsylvania Railroad Company; of the new water supply for DuBois, Pa.; and of improvements to the supplies of many other municipalities. In his work he was called upon to appraise the value of many existing water-works systems and to act as expert in many legal cases involving the diversion of water.

Those who were acquainted with Mr. Knight knew him to be a man of innate honesty and fairness and of a most genial nature, modest and unassuming. He never allowed his temper to get the better of him, nor did he show umbrage at provocation. His early experience as a contractor taught him fair dealing in his later work, and in all his engineering engagements his well formulated opinions carried great weight. Temperamentally, he was an optimist, always seeing the good and the bright side of life and its affairs, and a lover of the beautiful in human character and in Nature. He took a great interest in all matters pertaining to his native town and city.

Mr. Knight was a devout and consistent member of the Presbyterian Church, and was also a member of the Rome (N. Y.) Club.

He was twice married, his first wife having been Altay Elizabeth Potter, of Cleveland, N. Y., whom he married on October 18, 1875. Of the four children born of that marriage, one son, Arthur P. Knight, survives him. Mrs. Knight died on December 18, 1884. His second wife was Charlotte L. Lyndon, of Eaton, N. Y., whom he married on October 16, 1907, and who, with two children, Dorothy L. and Edgar Franklin, survives him.

Mr. Knight was elected a Member of the American Society of Civil Engineers on July 9, 1906.

JUDSON GILMAN TABLER, M. Am. Soc. C. E.*

DIED DECEMBER 28, 1924

Judson Gilman Tabler, the son of William D. and Mary (Reader) Tabler, was born on August 4, 1877, in Washington, D. C., where he received his education and grew to manhood.

Mr. Tabler was a Draftsman in the United States Patent Office at Washington when war was declared against Spain in 1898 and he promptly resigned his position to enlist. He served during that war with Company F of the 3d Volunteer Engineers and was in Cuba when mustered out of service. He remained on the Island until July, 1902, being engaged during this time as a Instrumentman on highway and railway construction.

In the summer of 1902, he returned to the United States and for the remainder of that year was located in Jennings, La., on irrigation work, going from there to Mexico in December to accept a position with the National Railways of Mexico. He remained in Mexico for more than eleven years, or until 1914, when the revolutionary state of the country had paralyzed all construction work. During this time Mr. Tabler rose from Instrumentman to the position of Chief Construction Engineer for the Company, having held many responsible and important positions in the interim. When forced out by the Revolution in 1914, he was engaged in building a railroad from Tampico to Vera Cruz, involving very difficult construction problems.

After his return to the United States, Mr. Tabler was employed for a time as Assistant Resident Engineer on maintenance of way by the International and Great Northern Railroad Company. In January, 1916, he became Principal Assistant Chief Engineer of the Berthe Engineering Company of Charleston, Mo., Consulting Engineers for various drainage and flood protection projects and other work. Except for a few months with the United States Engineers in Texas, in 1916, and a year spent in laying out and supervising the construction of a plant at Penniman, Va., including buildings, sewers, railroads, and yards, for the Dupont Powder Company, in 1917 and

1918, during the World War, he remained with the Berthe Engineering Company until June, 1921, when he resigned to accept a position with the Virginia State Highway Department. While with the Berthe Engineering Company, he had charge of construction work valued at more than \$3 000 000, including ditches, levees, pavements, flood-gates, sewers, and reinforced concrete and sub-foundation work.

Mr. Tabler remained with the Virginia State Highway Department until January, 1924, when he resigned to accept the position of County Engineer of Bedford County, Virginia, with headquarters at Bedford, where he had made many friends.

In the comparatively short span of life allotted to him Mr. Tabler had a wide and varied experience. The writer knew him intimately for more than twenty-seven years, having first met him in the Army in 1898, and afterward worked with him for years in Cuba, Mexico, and the United States. He was a man of real ability, at his best when conditions were most difficult, never losing his temper or his judgment in times of emergency and always ready to expend his last ounce of energy to "put the job through". Men who worked with him invariably not only admired and respected him as an Engineer, but valued him as a friend. He inspired true and lasting friendships among his fellow workers. Mr. Tabler was in every way the personification of an engineer and a gentleman and his memory will live long in the hearts of all who knew him. He was married in El Paso, Tex., to Dora M. Wrieth, who survives him.

He was a member of the American Association of Engineers, of the Masonic Fraternity, and of the Christian Church.

Mr. Tabler was elected a Member of the American Society of Civil Engineers on December 6, 1920.

TOBIAS TONNESEN, M. Am. Soc. C. E.*

DIED FEBRUARY 22, 1925

Tobias Tønnesen, the son of the Captain of the Harbor of Mandal, Norway, was born in Mandal on January 10, 1867.

After finishing school at home, he went at the age of 18 to Gothenburg, Sweden, to study engineering at the Chalmerska Institut of Technology. In his veins, however, ran the blood of the old Vikings and, in 1887, he left the Institute to embark for the United States and eventually landed at New York, N. Y.

At first he found it difficult to secure a position to his liking, but he finally succeeded in obtaining employment in a bridge shop, where he painted bridge girders. Not long afterward Mr. Tønnesen was promoted to a position in the Designing Department as a Draftsman.

* Memoir prepared by H. S. Hanssen, Esq., Marseilles, France.

In 1889 he went West, where he held several positions, first in the City Engineer's Office in Seattle, Wash., and, later, as a Topographer and Instrumentman on the Great Northern System. Subsequently, he became Assistant Engineer on the Everett and Monte Christo Railway; the California Midland Railroad, at San Francisco, Calif., and the San Francisco and Great Salt Lake Railroad. Mr. Tönnesen was also employed on irrigation and municipal work in the State of Washington; with the Nelson and Fort Sheppard Railroad, at Nelson, B. C., Canada; with the Great Northern Railroad; and also on the Portland, Ore., Water-Works.

Leaving the Northwest, Mr. Tönnesen went in 1895, to Guatemala, where he was engaged as an Assistant Engineer on the Ferro Carril Verapaz, Ferro Carril del Norte, and Ferro Carril Central. While thus employed he decided that it would be to his advantage to complete his theoretical knowledge of engineering and, therefore, he returned to Gothenburg in 1897 to enter again the Chalmerska Institut of Technology, from which he was graduated in 1899.

During his stay in Guatemala, Mr. Tönnesen had made the acquaintance of the owner of large coffee plantations, a Norwegian, Carlo Z. Thomsen, a resident of Hamburg, Germany, who introduced him to members of the large German Company, Otavi Minen und Eisenbahngesellschaft, of Berlin. This firm engaged him as a Consulting Engineer during the construction of its 350-mile narrow-gauge railway (0.60 m.) called the Otavi Railway, in South-West Africa, at the time the longest narrow-gauge railway in the world. The actual work was started in 1903, and was completed as far as the Tsumeb Copper Mine in 1906. During this period the Germans were at war with the Hereros.

On this railway Mr. Tönnesen had an opportunity to show his great ability and experience in railway construction. It was not only the longest, but also the most carefully constructed narrow-gauge railway in the world and gained wide renown. The contractors were Arthur Koppel and Company, of Berlin, which firm was also in charge of the location of the railway.

After the completion of the Otavi Railway, Mr. Tönnesen was engaged in 1907 as General Manager to the South-West Africa Company, Limited, of London, England, for its large estates and mining properties at Grootfontein in the northern part of South-West Africa. With the exception of a few years spent in Norway during the World War, when he was interested in mining in Sweden, he remained in this position until his death.

While in South-West Africa Mr. Tönnesen made numerous expeditions to its northern part to investigate the country, which was little known to white men, and he lectured on these expeditions and on the country before the Royal Geographical Society, London, to a large audience. On one of his trips he penetrated into Portuguese Angola, where he contracted sleeping sickness which ultimately caused his death. He died on shipboard on his way to his home in Oslo, Norway, on February 22, 1925, and was buried at sea. He is survived by his wife and two children, who live in Oslo.

As a memorial to him the South-West Africa Company, Limited, has erected at Grootfontein, a bronze bust, which stands on a 6-ft. base of Norwegian Laborador syenite. The bust was made by a young Norwegian artist.

Mr. Tønnesen was a real man, of strong opinions and principles, who did not hesitate to fight for what he thought to be right. He was unusually energetic and industrious, never sparing himself. He was gifted with common sense and had a wide, practical knowledge for which he was probably indebted to his experience in the United States, a country which he admired and often mentioned.

In addition to being the recipient of the Royal Norwegian Order of St. Olav, Mr. Tønnesen was also decorated with the Prussian Order of the Red Eagle for his services rendered in the construction of the Otavi Railway in South-West Africa.

Mr. Tønnesen was elected a Member of the American Society of Civil Engineers on May 1, 1901.

JOHN ROSS CHAMBERLIN, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 15, 1925

John Ross Chamberlin was born in Rochester, Ohio, on March 15, 1876. He was the son of Dr. Charles S. Chamberlin, a practicing physician, who died when John was five years of age. His mother, Tillie (Kisinger) Chamberlin, is still living.

Mr. Chamberlin received his early education in district schools and worked on the farm during vacations. When he was seventeen years of age the family moved to Tiffin, Ohio, and he entered the Academy of Heidelberg University in the fall of 1894. During the summer months he worked for the County Engineer, Mr. Charles J. Peters, and it was here that he acquired his first knowledge and experience as a civil engineer. In the fall of 1898, he entered the Ohio State University where he remained until April, 1902. He then withdrew from the University and went to Peru where he spent a year with the Engineering Corps in charge of the construction of the Cerro de Pasco Railway.

On his return to the United States in 1903, Mr. Chamberlin accepted a position with the Baltimore and Ohio Railroad, with headquarters at Baltimore, Md. In the fall of 1905, he returned to the Ohio State University as Instructor in Civil Engineering, and was, successively, Instructor and Assistant Professor of Civil Engineering until 1912. He was graduated from the Ohio State University with the degree of Civil Engineer as of the Class of 1902.

In 1912, he was appointed Division Engineer in the Bureau of Bridges of the Ohio State Highway Department, and, in 1913, Chief Engineer of Bridges, in which capacity he served until 1919.

* Memoir prepared by Clyde T. Morris, M. Am. Soc. C. E.

During this time Mr. Chamberlin was quite active in the study of the economics of highway bridge construction, and several *Bulletins* pertaining to bridge and pavement design were published by the Highway Department under his direction. In 1915 to 1918 he made a study of highway bridge specifications and, in 1918, the specification for highway structures of the Ohio State Highway Department was published. Many new and important provisions in this specification, especially with reference to reinforced concrete floors, and the determination of the required waterway for bridges, were the work of Mr. Chamberlin.

In October, 1919, he received an appointment as Senior Highway Bridge Engineer in the United States Bureau of Public Roads and spent a year in Washington, D. C. He was then transferred to District No. 5 at Omaha, Nebr., where he served until his death.

He was married in 1907 to Ada Maurer, of Chillicothe, Ohio, who, with five children, two sons and three daughters, survives him.

Mr. Chamberlin was appointed a member of the Special Committee on Concrete and Reinforced Concrete Arches of the Society in May, 1923. On learning of his death the following minute was signed by all the members of the Committee and sent to the Board of Direction:

"It was with deep regret that we, the members of the Committee on Concrete and Reinforced Concrete Arches of the American Society of Civil Engineers, learned of the death of John R. Chamberlin.

"Mr. Chamberlin was appointed on the Committee in May, 1923, and was a faithful and interested participant in its work until his death. He brought a wealth of good judgment and wise counsel to our work and his quiet courteous manner and lovable disposition endeared him to all.

"We feel keenly the loss of his good counsel in our proceedings and of his genial presence, and wish to express our sorrow at his loss, and our sympathy to his family which has lost a kind and loving father.

"(Signed) W. M. WILSON,

GEO. E. BEGGS,

E. H. HARDER,

A. C. JANNI,

CLYDE T. MORRIS."

Mr. Chamberlin was elected an Associate Member of the American Society of Civil Engineers on May 6, 1914.

JOHN STANTON GOODELL, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 13, 1925

John Stanton Goodell was born at Amherst, Mass., on January 20, 1875. He was the elder son of the late Henry Hill Goodell, formerly President of the Massachusetts Agricultural College, and Helen Stanton Goodell. He was descended from Puritan ancestors, the first of the family to come to America having been Robert Goodale who settled in Salem, Mass., in 1634. Eighty or more members of the family spelling the name either "Goodale" or "Goodell" served in the Revolutionary War.

* Memoir compiled from information on file at the Headquarters of the Society.

Mr. Goodell received his education at the Massachusetts Agricultural College, at Amherst, and, later, at the Rensselaer Polytechnic Institute, Troy, N. Y., where he completed his course in Civil Engineering.

From March 1, 1899, until February, 1908, he was connected with the Gulf, Colorado and Santa Fé Railway Company, in Texas, as Chainman, Rodman, Instrumentman, Levelman, and Transitman on locations at different times. He also served as Assistant Engineer on the Northern Division of the Railway, under F. Merritt, Division Engineer, at Clebourne, Tex., and as Office Engineer under C. F. W. Felt, M. Am. Soc. C. E., Chief Engineer, at Galveston, Tex.

In February, 1908, Mr. Goodell resigned, to accept a position with The Kwong Tung Yueh-Han Railway Company, Limited, at Canton, China, serving as Principal Assistant Engineer and, later, until August, 1910, as Engineer of Maintenance of Way, under the late Sir Chentung Liang Cheng, former Ambassador to the United States, who at that time was President of the Company.

From August, 1910, to October, 1911, he was engaged in private practice at Amherst, Mass., principally on land surveys, work on sewer lines, replacing lost highway monuments, etc. From October, 1911, to July, 1916, Mr. Goodell served as Manager of a rubber plantation in Nahiku, Island of Maui, Hawaii, and from July, 1916, to September, 1921, he was employed as Assistant Engineer in the Construction Department of the Atchison, Topeka and Santa Fé Railway Company, under C. F. W. Felt, Chief Engineer, at Chicago, Ill.

While in this position Mr. Goodell was engaged in checking estimates for proposed construction work in Missouri and later was in charge of a field office at Brunswick, Mo., where estimates, location and right-of-way maps, profiles, plans for the elimination of highway grade crossings, etc., were made for a proposed line from Standish to Moberly, Mo.

From July 21, 1917, to June 1, 1921, he was in charge of the construction of the Barton County and Santa Fé Railway from Holyrood to Galatia, Kans. On the completion of this work he returned to Amherst, Mass., where he engaged in private practice, continuing in this work until 1924 when his health became impaired.

Mr. Goodell was married on May 29, 1906, to Edith Friese, at Galveston, Tex., who survives him. He is also survived by his mother, Mrs. Helen S. Goodell, of Amherst, Mass., and a brother, Dr. William Goodell, of Springfield, Mass.

Mr. Goodell was elected an Associate Member of the American Society of Civil Engineers on March 7, 1906. He was also a member of the American Railway Engineering Association.

FRANK LAWRENCE SHELDON, Assoc. M. Am. Soc. C. E.*

DIED JUNE 22, 1925

Frank Lawrence Sheldon, the son of Frank Perry and Nellie (Noyes) Sheldon, was born in Providence, R. I., on August 1, 1883. He was a direct

* Memoir prepared by Frederick H. Paulson, Jun. Am. Soc. C. E.

descendant of Samuel Winsor, who married Mercy Williams, a sister of Roger Williams. His father, the founder of the firm of F. P. Sheldon and Son, was a noted mill engineer, having designed and re-organized many of the most prominent textile establishments in the United States.

Mr. Sheldon's earlier education was received in the public schools of Providence, and at Riverview Military Academy, Poughkeepsie, N. Y. In 1903, he entered Rensselaer Polytechnic Institute, Troy, N. Y., as a student in Civil Engineering, but left that institution in 1905, to enter his father's employ as Resident Engineer on mill construction, his first work having been on the buildings for the Lorraine Manufacturing Company at Pawtucket, R. I.

After serving for several years in this position, he entered the office of the Company and assisted in the design of various industrial plants. In 1912, he went to Knoxville, Tenn., to represent the firm of F. P. Sheldon and Son and to supervise the building of a large mill and weave shed for the Brookside Mills, under its plans. On the completion of this construction, he again took up office work, and, at the time of his death, was second in charge of the office, planning the work for the drafting force, checking plans and computations, and assisting in textile research.

Mr. Sheldon was a member of the Rensselaer Society of Engineers and of the American Society for Testing Materials, having served on its Committee on Textile Materials, as he was familiar with textile manufacturing and had made considerable research in this art.

Having been a proficient athlete in his school days—he had been a member of the football and hockey teams at Riverview Military Academy and at Rensselaer Polytechnic Institute—he always maintained a great interest in sports and was a keen student of modern football tactics. He was also fond of boating, hunting, and fishing, being at the time of his death a member of the Rhode Island Fish and Game Protective Association. Among the fine arts, music was his greatest delight, and while at Riverview Military Academy he was a member of the Glee Club.

Mr. Sheldon died, after a brief illness, at his home in Providence, on June 22, 1925, and is survived by his wife, Jetty L. (Wilson) Sheldon, to whom he was married on March 12, 1907, a son, Frank L., Jr., a sister, Mrs. James B. Barrett, and a brother, Arthur N. Sheldon, of the firm of F. P. Sheldon and Son.

Mr. Sheldon was elected an Associate Member of the American Society of Civil Engineers on June 6, 1921.